AMERICAN SEWERAGE PRACTICE

VOLUME I

DESIGN OF SEWERS

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BY

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AND

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CONTENTS

CHAPTER VIII

CHAPTER IX

CHAPTER X

CHAPTER XI

CHAPTER XII

CHAPTER XIII

CHAPTER XIV

CHAPTER XV

CHAPTER XVI

CHAPTER XVII

PREFACE

About three years ago the authors undertook the preparation of a book bringing together in a form convenient for ready reference the more important principles of theory and rules of practice in the design and operation of sewerage works, using this term in its broadest sense. It was found, however, that to make these fundamental data, tables, diagrams and rules of the greatest service, it was desirable to explain them in some detail, for such explanations can only be found scattered through many special treatises, transactions of technical societies, engineering journals and reports. In some cases it developed, to the authors' surprise, that nothing really definite had ever been published concerning many important features of sewerage practice. In other cases the practice of different engineers, being based upon their individual experiences, varied considerably. These conditions led the authors to broaden the scope of the work and to devote considerable space to topics upon which little had been written, in order that the reader might find all of the information which it was reasonable to expect in a comprehensive review of a subject of such scope as sewerage practice. It thus became necessary to present the subject in three volumes, the first dealing with the Design of Sewerage Systems, the second with their Construction, and the third with the Design of Works for the Treatment and Disposal of Sewage.

As the various chapters of this volume have developed much interest has been shown in the work by different engineers, both at home and abroad, who have supplied many helpful suggestions, valuable statements of their views upon subjects where experience furnishes a guide often more helpful than theory, and drawings of special structures to illustrate their standard practice in design. To these engineers hearty thanks are given for their cordial assistance in the authors' attempt to outline standard practice and sound principles of design.

The engineering journals have proved of valuable help, particularly in affording examples of practice and for their records of the development of present day methods; and many manufacturers have been most courteous in supplying drawings, photographs and specific information.

In the preparation of certain chapters of this volume, special aid has been obtained from "The Theory of Loads on Pipes in Ditches," by Professor Anson Marston and A. O. Anderson (Iowa State College of Agriculture and Mechanic Arts); "A Treatise on Concrete, Plain and Reinforced," by Frederick W. Taylor and Sanford E. Thompson (John

Wiley & Sons); "Principles of Reinforced Concrete Construction," by Professors F. E. Turneaure and E. R. Maurer (John Wiley & Sons); "A Treatise on Hydraulics," by Professor Hector J. Hughes and Arthur T. Safford (copyright, 1911, The Macmillan Company); "American Civil Engineer's Pocket Book," edited by Mansfield Merriman (John Wiley & Sons); and "A Treatise on Masonry Construction," by Professor Ira O. Baker (John Wiley & Sons). While acknowledgment has been made in the several chapters, for this help, more specific thanks are here given for the generous permission to make such free use of these valuable contributions to engineering literature. The authors have also drawn upon the late August Frühling's valuable "Entwässerung der Städte," published by Wilhelm Engelmann.

The authors are under obligations to their junior partners, Charles W. Sherman, William T. Barnes and Almon L. Fales, and to their office staff, particularly William L. Butcher and Frank A. Marston, for valuable assistance in the preparation of this book, and to John M. Goodell, for many years editor-in-chief of "Engineering Record," whose constructive criticism and assistance in the preparation of the manuscript have been most helpful. To the publishers, the McGraw-Hill Book Company, Inc. whose work speaks for itself, thanks are also given.

Whatever its merits or demerits, the book is at least a monument to co-operative effort and good will among civil engineers.

The preparation of this book has demanded an amount of time and effort far in excess of that anticipated when the work was undertaken. The authors have carried it through, however, because of their experience of the practical value of such information as is given in many chapters herein. As problems have arisen in their work, reference has been made to the book for the help required, and if anything was found lacking it was supplied. This practical test has resulted in the repeated revision of large portions of many of the chapters. The book is published, therefore, with the belief that it is a "practical" book, but as the test of service in one office is not a thorough test of a book on American Sewerage Practice, considered comprehensively, the authors will be glad to receive from the reader any suggested additions, changes or modifications which will make the book more helpful, and to have any errors of statement or computation called to their attention.

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AMERICAN SEWERAGE PRACTICE

INTRODUCTION: THE LESSONS TAUGHT BY EARLY SEWERAGE WORKS

American sewerage practice is noteworthy among the branches of engineering for the preponderating influence of experience rather than experiment upon the development of many of its features, apart from those concerned with the treatment of sewage. Even the actual capacity of sewers, something that gagings can determine, is far less clearly known today than is the capacity of water mains, while the cross-sections of masonry sewers and the forms of accessory structures employed under similar conditions in different cities vary widely. There has been, however, a rather decided tendency toward greater uniformity in design in the last ten to fifteen years, keeping abreast with the growing popular recognition of the financial and sanitary importance of good sewerage and the passing of the feeling that it was a bit indelicate to speak in public of anything so unclean as sewage. Sewerage systems, being out of sight, were out of mind, except to the few intrusted with their construction and maintenance, and even today the lack of anything above ground to show to the taxpayer makes sewerage work in a city one of its least appreciated activities. The strong feeling that good public health is a valuable municipal asset and depends to a large extent upon good sewerage has been a leading cause of the willingness of taxpayers recently to embark on expensive sewerage undertakings.

The value of arousing public feeling toward sewerage in this way is the main lesson which the history of sewerage teaches. Until it became a strong influence, sewerage work was concerned mainly with surface drainage and the abatement of nuisances. The first record of a sewer which Curt Merckel, the antiquarian of engineering, has been able to find is on an old Babylonian seal-cylinder. Layard's explorations revealed arched sewers in Nineveh and Babylon dating from the seventh century before Christ. Schick and Warren have unearthed considerable information about the sewers of Jerusalem, the works of this class in Grecian cities are fairly well known, and the great underground drains of Rome have been repeatedly described. We know, however, that these channels and conduits were not used to any extent by means of direct connections to them from the houses, for the requirements of public health were little recognized then and compulsory sani-

tation would have been considered an invasion of the rights of the individual. Livy states that the Roman building regulations only stipulated that the house connections were to be made at the cost of the property owners. Public latrines were doubtless used by most of the people and it is probable that the gutters were the chief receptacle of the ordure of the city, which was washed thence into the sewers. These must have been extremely offensive when not flushed, for otherwise the regular delivery of water for the purpose of cleaning them would not have been so emphasized in the following notes by Frontinus, a water commissioner of the city whose valuable notes of engineering work have been edited by Clemens Herschel:

"I desire that nobody shall conduct away any excess water without having received my permission or that of my representatives, for it is necessary that a part of the supply flowing from the water-castles shall be utilized not only for cleaning our city but also for flushing the sewers."

It is astonishing to reflect that from the day of Frontinus to that of W. Lindley,[1] there was no marked progress in sewerage. The renaissance began in Hamburg, where a severe conflagration destroyed the old part of the city in 1842. The portion ruined was the oldest section and it was decided to rebuild it according to modern ideas of convenience. This work was intrusted to Lindley, who carried it out in a way that aroused warm praise among engineers of a somewhat later period, when the test of service had placed the seal of approval on the plans.

For instance, E. S. Chesbrough, Moses Lane and Dr. C. F. Folsom reported to the authorities of Boston in 1876 that Hamburg

"was the first city which had a complete systematic sewerage system throughout, according to modern ideas. How far that was in advance of the rest of the world, in 1843, when the work was undertaken, may be inferred from the fact that there are no real advances in new principles from that time up to the present day. The rain-water spouts were all untrapped to serve as ventilators to the sewers; the street gullies were also without traps, and there were gratings for ventilation opening into the streets. It is very rare that any of the latter are sources of complaint, inasmuch as the sewers are kept so clean that there are seldom any foul-smelling gases. The great feature in Hamburg, however, is the weekly flushing at low tide by letting the waters of the Binnen-Alster flow through the sewers with great force."

Twenty-five years after the sewers were completed they were found by a committee of experts to be clean and almost without odor.

The sewerage of Hamburg, while indicative of an awakened public recognition of the need of improvement in such works, was hardly the result of any real appreciation of the value of sanitation but was rather

[1] Lindley was one of the leading English engineers of his day, Rawlinson being his only rival at the head of the sanitary branch of his profession. He became thoroughly identified with German work, however, first at Hamburg and later at Frankfort.

the result of business shrewdness in taking advantage of exceptional local conditions to plan streets and sewers to answer in the best way the recognized needs of the community and the topographical conditions. The history of the progress of sanitation in London probably affords a more typical picture of what took place about the middle of the nineteenth century quite generally in the largest cities of Great Britain and the United States.

A statute was passed in 1531 in Henry VIII's reign[1] and amended in that of William and Mary which afforded the legal basis of all sanitary works of sewerage well into the nineteenth century. For a period of about 300 years, while London outgrew the narrow limits of the City proper and its adjacent parishes and became a great metropolis, the center of the world's commerce, sanitation was as little considered as magnetism or the utilization of steam for power purposes. The City was better off than most of the metropolitan district, for it had Commissioners of Sewers elected annually by the Common Council from its members. They had power over all conditions relating to public health and comfort and had authority to appoint a medical officer of health. But the City was only a small part of the metropolitan area, 720 out of 75,000 acres in 1855, with only 128,000 out of a total population of 2,500,000, and less than 15,000 out of a total of 300,000 houses. Outside of the City, the methods of local government were chaotic; in some of the parishes surveyors of highways were appointed to do very restricted engineering work, and in eight there were Commissioners of Sewers, apparently having powers modeled after those of the City but less extensive.

This lack of central authority rendered a systematic study and execution of sewerage works impossible. As late as 1845 there was no survey of the metropolis adequate as a basis for planning sewers. The sewers in adjoining parishes were on different elevations so that a junction of them was impracticable. "Some of the sewers were higher than the cesspools which they were supposed to drain, while others had been so constructed that to be of any use the sewage would have had to flow uphill. Large sewers were made to discharge into small sewers." (Jepson's Sanitary Evolution of London.) The first engineer to make a comprehensive study of metropolitan sewerage needs in an official capacity, John Phillips, gave this testimony[2] of the condition of London basements and cellars in 1847:

[1] Statues relating to local drainage problems had been passed in the reigns of Henry III, Henry VI and Henry VII.

[2] Lest this picture be considered too fanciful, a statement published in 1852 by the General Board of Health may be quoted here;

"During the first labors of the General Board of Health much illness prevailed among the clerks, until on one occasion foul smells arising more severely than had before been noticed, the state of the foundations was examined, when it was discovered that there were two very large cesspools immediately beneath the Board's offices. This is the description

"There are hundreds, I may say thousands, of houses in this metropolis which have no drainage whatever, and the greater part of them have stinking, overflowing cesspools. And there are also hundreds of streets, courts and alleys that have no sewers; and how the drainage and filth are cleaned away and how the miserable inhabitants live in such places it is hard to tell.

"In pursuance of my duties, from time to time, I have visited very many places where filth was lying scattered about the rooms, vaults, cellars, areas and yards, so thick and so deep that it was hardly possible to move for it. I have also seen in such places human beings living and sleeping in sunk rooms with filth from overflowing cesspools exuding through and running down the walls and over the floors. . . . . The effects of the effluvia, stench and poisonous gases constantly evolving from these foul accumulations were apparent in the haggard, wan and swarthy countenances and enfeebled limbs of the poor creatures whom I found residing over and amongst these dens of pollution and wretchedness."

One of the main reasons for the backward condition of the sewerage system in London for many years was the absence of authority to compel landlords to connect their houses with sewers, so that even the residences of the wealthiest members of the nobility were likely to be located over one or more cesspools, some of which were occasionally of enormous size. Even in Westminster, very little use was made of the sewers in some of the streets. "So long as the owners get the rent, they do not care about the drainage," the Commissioners of Sewers reported in 1845. It was not until two years later that the first act was passed making it compulsory to connect houses with sewers.

In 1847, scared by an outbreak of cholera in India, which had begun to work westward, a royal commission was appointed to inquire into sanitary improvements for London. This body reported that the sewerage of the entire metropolitan district should be handled by a single board, and in 1848 Parliament followed this advice and created the Metropolitan Commission of Sewers. That body and its successors in the office unfortunately failed to measure up to their opportunities; they produced reports showing clearly the need of extensive sewerage works and other sanitary improvements, built the Victoria sewer at great expense, which fell into ruins not many years later, but did little more. In the summer of 1848 cholera was discovered in London and before the winter was over it claimed 468 victims. It broke out again in the spring of 1849 and before it ended about 14,600 deaths were recorded, as against 6729 in London in the 1832–33 epidemic.

In 1852 cholera again appeared and in 1853 it slowly gained a foot-

of houses of which it is generally reported by house agents and others that they are well drained and in good condition; but it may be advised that it is absolutely unsafe to take any house without a thorough examination of the site beneath it, nor where any cases of fever, typhoid or gastric, have occurred amongst persons living in the lower offices of a house, is it safe for those who value their own health to remain in the premises without such an examination, nor until the cesspools are removed."

hold. In 1854 it ran its terrible course, claiming a mortality of 10,675 in the last half of that year. The connection between a contaminated water supply and the rapid spread of the disease was clearly shown, but it was also apparent that the filthy living conditions in most houses, due to the absence of effective sewerage, was a great hindrance in combatting the scourge. In 1855 Parliament passed an act "for the better local management of the metropolis;" this laid the basis for the sanitation of London and provided for the Metropolitan Board of Works which soon after undertook an adequate sewerage system.

In this connection a brief mention of some of the features in the early development of the London sewers will be of value as showing by contrast the importance of the progress in sewerage in recent years.

In 1849, in answer to an advertisement, the Metropolitan Commission of Sewers received 116 different schemes for abating the nuisance due to sewage in the Thames; none was approved for execution. Plans for intercepting the sewage and conducting it to outlets below the city had been suggested many years before. It was not until 1852, when J. W. Bazalgette became chief engineer of the Commission, that any beginning was apparently made in formulating policies, although at least two engineers of high standing connected with the local works of certain subdivisions of the metropolitan district had been making valuable studies. Bazalgette seems to have been possessed of the executive ability previously lacking; he developed plans for interception tentatively and then worked them out in detail in collaboration with W. Haywood, the unusually gifted and highly respected engineer of the City Commissioners of Sewers, who had a thorough local engineering experience and was responsible for many of the basic assumptions upon which the plans were prepared. But no action was taken on these plans until the Metropolitan Board of Works appointed Bazalgette its engineer and he had been compelled to uphold them against lay and engineering criticism for several years. The works were not actually undertaken until 1859.

The old sewers were frequently the covered channels of brooks. The oldest was Ludgate Hill sewer, of unknown age, but built prior to Fleet Street sewer, which was constructed in 1668 and was an open channel for many years. This sewer, formerly known as the River of Wells or the Old Bourne (now called Holborn), was fed by several springs, and was originally a navigable waterway from which people were supplied with water. It was not covered until 1732. What eventually became Ranelagh sewer was a brook rising in a spring at Tye Bourne and even as late as 1730 it furnished water for the Serpentine, the famous pond in Hyde Park. In 1855 the total length of these old sewers was 1146 miles. Up to 1815 it was contrary to law to discharge

sewage or other offensive matter into the sewers; cesspools[1] were regarded as the proper receptacles for house drainage and sewers as the legitimate channels for carrying off surface water only. The cesspools were cleaned by private contractors at the expense of the property owners, and consequently the frequency of the cleaning depended on the callousness of the owner or tenant to complaint and nuisance. Concerning the removal, the General Board of Health reported in 1852 as follows:

"It appears that the quantity of cesspool refuse, including ordure and other animal and vegetable matter, is from 1 to 2 cu. yd. per house per annum; and the cost of its removal in London (including openings and making good the cesspool, and cartage out of town) was stated by contractors, and proved upon a house-to-house inquiry, to be on an average, about 20s. per house. When cheap cesspools are made, from which percolation is not prevented, the injury to the foundations of the houses would more than make up the difference. In many country towns, where night-soil is kept in shallow uncovered pits (called midden-holes) the cost of emptying is less than where deep cesspools are used, but although the emanations, as being more diluted, may be less noxious than those arising from covered cesspools, the sight of the exposed ordure is offensive and degrading, and the open midden-steads are in other respects serious nuisances."

It has been mentioned that the pollution of the Thames was a cause of public protest in the middle of the last century; it was aggravated by the manner in which the sewers discharged their contents. Bazalgette's description of it is worth quoting as explaining a feature of outfall sewer design which is sometimes overlooked at the present time:

"According to the system which it was sought to improve, the London main sewers fell into the valley of the Thames, and most of them, passing under the low grounds on the margin of the river before they reached it, discharged their contents into that river at or about the level, and at the time only, of low water. As the tide rose, it closed the outlets and ponded back the sewage flowing from the high grounds; this accumulated in the low-lying portions of the sewers, where it remained stagnant in many cases for 18 out of every 24 hours. During that period, the heavier ingredients were deposited, and from day to day accumulated in the sewers; besides which, in times of heavy and long-continued rains, and more particularly when these occurred at the time of high water in the river, the closed sewers were unable to store the increased volume of sewage, which then rose through the house drains and flooded the basements of the houses. The effect upon the Thames, of thus discharging the sewage into it at the time of low water, was most injurious, because not only was it carried by the rising tide up

[1] "What are termed dry wells in the United States differ from the London cesspools in this particular; they consist of excavated pits in the subsoil, sustained by pervious masonry linings, and are not intended to be cleaned out until the surrounding earth fails to absorb their contents; while the London cesspools were constructed with impervious masonry linings and were designed to be cleaned out at proper intervals." (Report by Charles Hermany on Memphis sewerage, 1868.)

the river, to be brought back to London by the following ebb-tide, there to mix with each day's fresh supply—the progress of many days' accumulation toward the sea being almost imperceptible—but the volume of the pure water in the river, being at that time at its minimum, rendered it quite incapable of diluting and disinfecting such vast masses of sewage."

In designing the great intercepting and outfall sewers to remedy this condition, Bazalgette adopted a mean velocity of 2.2 ft. per second as adequate to prevent silting in a main sewer running half full, "more especially when the contents have been previously passed through a pumping station." The computation of the house sewage was based on an average density of population of 30,000 persons per square mile except in the outlying districts, where it was assumed at 20,000. The sewage was estimated at the assumed water consumption, 5 cu. ft. per capita daily. "This quantity varies but little from the water supply with which a given population is provided; for that portion which is absorbed and evaporated is compensated for by the dry-weather underground leakage into the sewers." One-half of this sewage was assumed to flow off within 6 hours. The storm-water run-off[1] for which provision was made was a rainfall at the rate of 1/4 in. per day received during the 6 hours of maximum sewage flow, with overflows to discharge the excess due to larger amounts through some of the old sewers directly into the river.

It is not surprising, in the light of present information summarized in one of the following chapters, that these estimates proved too low and flooding took place in low-lying districts. As for the average

[1] "There are, in almost every year, exceptional cases of heavy and violent rain storms, and these have measured 1 in., and sometimes even 2 in., in an hour. A quantity equal to the 1/100 part of an inch of rain in an hour, or 1/4 of an inch in 24 hours, running into the sewers, would occupy as much space as the maximum prospective flow of sewage to be provided for; so that, if that quantity of rain were included in the intercepting sewers, they would, during the 6 hours of maximum flow, be filled with an equal volume of sewage, and during the remaining 18 hours additional space would be reserved for a larger quantity of rain. Taking this circumstance into consideration, and allowing for the abstraction due to evaporation and absorption, it is probable that if the sewers were made capable of carrying off a volume equal to a rainfall of 1/4 in. per day, during the 6 hours of the maximum flow, there would not be more than 12 days in a year on which the sewers would be overcharged, and then only for short periods during such days." Bazalgette, Proc. Inst. C. E., xxiv, 292.

"The total sewage and rainfall provided for was 394,000,000 imp. gal. per day. The discharging capacity of the sewers was, however, made larger than this amount, as it is a well-known fact that, owing to the fluctuating flow of sewage at different hours of the day, about one-half of the total quantity flows off in 6 hours, and as figures in the above tables [108,000,000 imp. gal. of sewage and 286,000,000 imp. gal. of rain-water.—M. & E.] give the flow of sewage spread over the whole 24 hours, provision had to be made and was made for practically double the amount of sewage given in these tables. . . This provision for discharging excessive rainfall into the Thames by means of the old sewers could not be satisfactory at all times, as it has already been pointed out that these old sewers were blocked by the tide for a considerable time before and after high water, and, therefore, the rainfall could only reach the Thames at some time on each side of low water, unless in any case the old sewers were capable of being put under such a pressure as would overcome the opposing pressure of the tidal head." Maurice Fitzmaurice, "Main Drainage of London."

minimum velocity selected, it was higher than that recommended by some contemporary engineers. Wicksteed had reported experiments showing that a bottom velocity of 16 in. per second would move heavy pieces of brick and stone, and a velocity of 21 3/4 in., would move iron borings and heavy slag. John Phillips advocated a velocity of 2 1/2 ft. per second. Professor Robison said in his "Theory of Rivers" that a bottom velocity of 3 in. per second will take up fine clay such as potters use, 6 in. will lift fine sand, 8 in. will lift sand as coarse as linseed, 12 in. will sweep along fine gravel, 24 in. will roll along 1-in. pebbles, and 36 in. will move angular stones of the size of an egg. These statements of the state of knowledge in 1850 show a tendency to underestimate requisite velocities to prevent silting, and, taken in connection with the underestimates of run-off and their unpleasant consequences, illustrate the great desirability of adequate experiments to ascertain unknown facts essential for successful design, before spending great sums on construction.

The questionable character of the information available for design was recognized by a number of engineers, as the following remarks[1] by Sir Robert Rawlinson indicate clearly:

"To talk of a formula for main sewers, devised and drawn up from any one set of experiments, would only tend to mislead young engineers. There were no two places which required precisely the same treatment. . . . The proper mode of proceeding was: before attempting to fix the dimensions of main sewers, to take the area to be operated upon as it existed; to consider what nature had previously done with that area; then to consider the special duties which the sewers had to perform, and apportion them to the water supply and to the probable increase of the population; and if the dimensions adopted were calculated for passing off three times or four times that volume, the Engineer would not be far astray in his calculation" (Proc. Inst. C. E., xxiv, 317).

Prior to Haywood and Bazalgette's work on the London intercepting sewers, Phillips and Roe were prominently before the public as sewerage experts and among English-speaking engineers Roe's Table[2] was used for many years in selecting the sizes of sewers. As a matter of historical interest, for it was used by many early American engineers, it is reproduced in Table I. It was acknowledged to be entirely empirical

[1] It is evident that Sir Robert was speaking of sewers for house drainage only. He was the leader in the development of modern sewerage practice and exercised a great influence over the engineers of his day and the public.

[2] Roe's Table was not accepted by some contemporary London engineers, and in 1865 W. Haywood, engineer of the City, who remained for half a century a leading authority on English municipal engineering, stated at a meeting of the Institution of Civil Engineers that there were no reliable gagings of London sewers in existence and that he had never been able to obtain any accurate information regarding such work from either Phillips or Roe. He stated that he had been forced to make extensive gagings in consequence, and these showed that about half the sewage coming daily from the 11 square miles tributary to the gaging stations passed off between 9 A. M. and 5 P. M.

and was based on Roe's observations in the Holborn and Finsbury divisions of the London sewers during more than 20 years, he said. Roe once stated in some testimony that the particulars from which the table was compiled filled upward of a hundred memorandum books. In some cases the records were asserted to relate to observations carried on during the whole period of heavy rains, being commenced as each storm began and continued until its effect had ceased in the sewers, the depth of water being taken every 5 minutes and the velocity of current noted at every depth. In some instances the observations were continued day and night for 2 years and in others for periods of a few

TABLE 1.—ROE'S TABLE, SHOWING THE QUANTITY OF COVERED SURFACE FROM WHICH CIRCULAR SEWERS (WITH JUNCTIONS PROPERLY CONNECTED) WILL CONVEY AWAY THE WATER COMING FROM A FALL OF RAIN OF 1 IN. IN THE HOUR, WITH HOUSE DRAINAGE.

Diameter of sewer, inches	24	30	36	48	60	72
	acres	acres	acres	acres	acres	acres
Level	38¾	67½	120	277	570	1,020
1 in 480	43	75	135	308	630	1,117
1 in 240	50	87	155	355	735	1,318
1 in 160	63	113	203	460	950	1,692
1 in 120	78	143	257	590	1,200	2,180
1 in 80	90	165	295	670	1,385	2,486
1 in 60	115	182	318	730	1,500	2,675

Diameter of sewer, inches	84	96	108	120	132	144
	acres	acres	acres	acres	acres	acres
Level	1,725	2,850	4,125	5,825	7,800	10,100
1 in 480	1,925	3,025	4,425	6,250	8,300	10,750
1 in 240	2,225	3,500	5,100	7,175	9,550	12,400
1 in 160	2,875	4,500	6,575	9,250	12,300	15,950
1 in 120	3,700	5,825	7,850	11,050	14,700	19,085
1 in 80	4,225	6,625	....	....	....	....
1 in 60	4,550	7,125	....	....	....	....

Note.—By "house drainage" Roe meant rain-water from roofs and courts. It is difficult to give the date of the first publication of this table, but it was probably between 1840 and 1845.

months in several years. Roe laid much stress on curving all junctions, preferring a 30-ft. radius; where the junctions are made at right angles he advised using larger sewers than those given in the table, a recommendation that seems to have been generally overlooked. Another thing to be considered in using the table he stated thus:

"In applying the table to localities where the inclination is greater than that of the Holborn and Finsbury divisions, a modification of the sizes of the sewers will be required; for instance, in one case that came under my notice, when the general inclination of the surface of the streets was about 1 in 20, the greatest flow of water from a thunderstorm came to the sewer at the rate of one-third more than it did to a sewer draining a similar fall of rain from an area with a general surface inclination of 1 in 132."

It should be said here that sewerage progress elsewhere in England was apparently less opposed than in London. In 1848 Parliament passed a sanitary code applying to all parts of England and Wales except London, and in 1855 it enacted a nuisance removal law for all England; these laws were the basis of the subsequent sanitary progress outside the metropolis for many years. It will be observed, however, that the development of sewerage undertakings in that country was a direct result of the awakening of the people by a succession of epidemics of choléra, for progress did not begin until that disease had twice terrorized the country within a short period.

The present sewerage system of Paris, like that of London, was inaugurated as a result of a cholera epidemic. The system is unique in some ways, although in its early days the Parisian sewers were doubtless little different from the conduits enclosing old brooks or receiving storm water which were constructed in many large cities. The Menilmontant sewer, mentioned in a record of 1412, was of this type, and remained uncovered until about 1750. It intercepted the drainage of the streets on the northern slope of the city's area lying on the right bank of the Seine and was called the "great drain" (*grand égout* or *égout de Ceinture*). The part of the city on the left bank of the river was drained by open gutters leading down the centers of the streets to the river.

The first attempt to study the sewerage needs of the city comprehensively was apparently made in 1808, when there were 14 1/2 miles of drains with about 40 outlets into the river, and during the next 24 years about 10 1/2 miles more of drains were constructed. In 1832 the ravages of cholera awakened the authorities to a partial realization of the city's unsanitary condition. The following year a topographical survey was made, and with the aid of the maps based upon it, five systems or divisions of sewerage were planned, based on topographical features of the territory rather than on the administrative boundaries of parishes, which caused so much delay in the development of rational drainage at London and have been harmful in the United States. Many of the low-lying streets along the river were raised at this time above the level of any known flood, which indicates that the work was regarded as drainage rather than house sewerage. The regulation of the streets was attended in some cases by the reconstruction or entire abandonment of the old sewers in them. One of the most interesting features of the work

was the change in the cross-section of the streets from concave to convex, for reasons explained by H. B. Hederstedt:

"With regard to the conversion of the concave surfaces of roads into convex, it may be shown to have formed an important part of the drainage system. Against hollow roads, there were always complaints. The old plan constantly cut up the roadway with cross-channels or gutters. Another object had been considered, however, in making the change, the certainty of freeing the roads more readily from rainfall. In the concave roads, iron gratings were set on the top of small working shafts, built on the crown of the drain-arch. These iron gratings frequently became clogged and the passage of the water was impeded to such an extent that raised planks were occasionally used to enable foot passengers to cross the road, the vehicles meanwhile being compelled to travel through a sea of mud. The old roadway had, in many places, to be lifted to obtain sufficient headway for the minimum-sized drains; the value of the convex roads, as affording an extra height, is therefore obvious." (Proc. Inst. C. E., xxiv, 262).

The new sewers built in Paris from 1833 onward were made 6 ft. or more high wherever possible, in the belief that the workmen employed in cleaning them would discharge their duties more efficiently if they could labor without being forced to take unnatural positions.[1] Toward 1848 the little sewers were given a minimum height of 5.5 to 5.9 ft. without exception, and a width of 2.3 to 2.6 ft. at the springing line of the arch, the width at the invert being a trifle less. "These dimensions are too scanty; for getting about easily at least 2 m. height and 1 m. width are needed." (Humblot.) When it became necessary later to enlarge some of these small sections to receive water mains, the top was widened out on one side (sometimes on both sides) while the lower part was left narrow, thus producing those sections shaped something like the letter P which have been the subject of strange comments from persons unfamiliar with their origin.

Although there has been a great deal of criticism of the large Parisian sections it has generally failed to take into account that the sewers of that city have been built with a view to removing street refuse as well as sewage and rain-water. There are no catchbasins on these great drains so that everything entering the inlets, and not caught in the little baskets suspended in some of them, passes directly into the sewers.

[1] "One reason for making the smallest class of public sewers in Paris so much larger than they are in every other city is the practice which, till within 10 years, existed only there, of placing the water mains in them." (E. S. Chesbrough, 1856.) In commenting on the location of water mains in the sewers, Humblot stated in a report in 1886 that the flat portions of Paris were largely on filled ground and the hills were undermined by old quarries, so that leaks in mains laid in earth would rarely be detected, and it was particularly desirable to keep the mains exposed so that their condition could be observed constantly. Telegraph and telephone lines and pneumatic tubes for transporting mail were placed in the sewers on account of the facility of installation and maintenance. Gas mains were also placed in a few of the sewers until explosions led to the abandonment of this practice.

The streets are cleaned largely by washing them with hose streams. The street litter is flushed into the sewers and is swept down the latter by storm water and by the sewer-cleaning gangs into the larger sewers or collectors. Some of the sewer sludge is removed through manholes but most of it is flushed through the collectors or intercepting sewers by the *bateaux vannes* and *wagons vannes.* These are boats or cars provided with wings reaching nearly to the walls of the channels. The wings dam up the sewage somewhat and it escapes around their edges with a higher velocity than that of the ordinary current. In this way the sludge is stirred up and carried along ahead of these cleaning devices. Other means of cleaning are also employed, but it is unnecessary to describe them here; the reader interested in the subject will find the whole field of the design, construction and management of the Paris sewer system described in Hervieu's "Traité Pratique de la Construction des Égouts" (Paris, 1897). A large part of the sludge is forced along into large chambers on the banks of the river, where it is discharged through chutes into barges which remove it to various places of disposal.

The chief feature of this work inaugurated in 1833 was its recognition of the principle of interception. Longitudinal drains of large section were laid out parallel to the river and only three of the forty old mouths of independent sewers were left in service, the remaining systems being made to discharge into the intercepters. The rain-water falling on the roofs was taken at first through leaders to the gutters, but later was diverted in some cases to the large "house drains," with sections big enough for a man to walk through, connecting the houses with the sewers but used only for delivering waste water and not for excrementitious matter. The latter was discharged for many years into cesspools, one frequently answering for an entire block of houses.

"The Parisians committed the fatal mistake about 1820, of insisting by ordinance on cesspool construction. It was recorded that the whole subsoil of Paris was on the point of becoming putrid with cesspit matter, and that the ordinance was passed in consequence. By it all cesspits, as matters of private construction, were abolished, and the construction of cesspools on a gigantic scale was undertaken or directed by the municipality, and all persons thereafter building houses were obliged to construct 'hermetically-sealed cesspools' after a municipal or royal plan which had been devised by the government engineers of France. Into those cesspools effete matter from water-closets, grease and washings from the sinks, and such refuse was to be discharged." (Sir Robert Rawlinson, Proc. Inst. C. E., xxiv, 318.)

The cesspools finally became so offensive that the nostrils of the Parisians were plagued and a new system of sewerage was accordingly developed. At that time European sanitarians were divided into two schools, advocating respectively the "dry" and the "water carriage" methods of collecting excrementitious matter. In the former this matter

is collected and removed in pails and in the latter it is flushed into the sewers. The former is still used in a number of European cities, but as it is not employed in the United States it is necessary here to give only the following brief account of it, abridged from Dr. Hering's report on European sewerage, mentioned later in this chapter. A complete summary of the subject is given in Baumeister's "Cleaning and Sewerage of Cities," where the methods of cleaning cesspools, the equipment for "dry" collection, and the disposal of the contents of the pails are treated with a detail unnecessary to repeat for American readers.

Water carriage was opposed by European chemists, physicians and agriculturalists because of a fear of contamination of the soil by leakage from the sewers, the possible pollution of bodies of water receiving the sewage and possible nuisances if not actual dangers where the sewage was distributed over land. Engineers were generally favorable to water carriage. Dr. Pettenkofer, the famous hygienist, was at first an opponent of it but subsequently became an advocate.

Dry removal accomplished its object satisfactorily, either by an immediate and thorough disinfection with subsequent removal at convenient intervals or by temporary storage with frequent removal before decomposition could be rendered injurious.

There were two common methods of disinfection. The first was the partial absorption of the sewage by dry earth, peat, charcoal and like materials, which accelerated its decomposition and diminished offensive odors. The second was the addition of carbolic acid, chloride of lime, creosote oil and other chemicals to the sewage.

Where there was no disinfection, the excreta were collected in a "pail," (called *fosse mobile* in France and *tonne* in Germany) made of iron or oak and provided in some cities with a tight lid having a sleeve fitting closely around the bottom of the soil pipe. These pails were collected at intervals of a day to a week and clean ones substituted for them. Where the system was conducted most satisfactorily, the pails were removed in wagons with tightly closed bodies and were carefully cleaned after being emptied. The contents were frequently used for fertilizing purposes.

The dry system, to compare favorably with the water carriage system, must be restricted to (1) small towns, on account of the expense of cartage; (2) towns where the regular exchange of the pails can be enforced with almost military strictness, which is seldom found outside of a few European countries; (3) dwellings where water-closets cannot be used; (4) localities where sewerage would be very expensive; (5) where the waste water can be led over the surface of the ground without causing offense.

There was an unusual modification of the pail system employed for some time in Paris after the cesspools became too offensive. The engineers of the city were early advocates of water carriage for removing fecal matter, but there was great popular opposition to this although the large storm-water sewers were available for water carriage and their contents were already foul with the refuse washed from the streets. Accordingly a *fosse filtre* was temporarily used to educate the public. It was a cask of 20 to 25 gal. which

retained all solids reaching it through the soil pipe but permitted the escape of the liquids into the sewer. As the liquids are the most putrescible parts of the excreta, some sanitary gain was made in this way, and as soon as popular prejudice abated the pail and its connections were removed and the soil pipe connected with the house drain by a few feet of pipe.

The early sewerage works in the United States are almost unknown.[1]

[1] The early American sewerage engineers of note were first engaged on such work by chance, not inclination. The list is headed by E. S. Chesbrough, who was born in 1813 and died in 1886. He became a chainman on railroad surveys when he was 15 years old, and rose gradually in railroad engineering positions until 1846, when he became chief engineer of the Western Division of the Boston water works. He was reluctant to accept this work on account of his lack of familiarity with anything but railroad engineering, and only undertook it with the assurance that J. B. Jervis would act as consulting engineer. He remained on this work until he became city engineer of Boston, in 1850, and thus first became interested in sewerage. He resigned in 1855 to become the engineer of the Chicago Sewerage Commission and while holding this office he published in 1858, a voluminous report on sewerage which was the first really important American exposition of the subject. His plans for the Chicago sewers were adopted and that city was the first important place in the country to engage on the systematic execution of a comprehensive sewerage system. This established his reputation as a specialist and he was subsequently consulted in connection with sewerage problems by Boston, Burlington, Ia., Chattanooga, Des Moines, Dubuque, Memphis, New Haven, Peoria, Providence and many smaller places. He was the eighth president of the American Society of Civil Engineers.

Moses Lane, like Chesbrough, was a railroad engineer in early life. He was born in 1823 and was graduated from the University of Vermont in 1845 as a civil engineer. He was engaged in alternating periods on railroad engineering and as a teacher down to about 1857, when he became principal assistant engineer of the Brooklyn water works, under J. P. Kirkwood, and finally succeeded him. In 1869 he became a partner of Chesbrough in Chicago and thus came into touch with sewerage work for the first time. His most important plans for sewers were the systems for Milwaukee and Buffalo, but he also furnished plans for a number of smaller places. When he died in 1882, he was serving as city engineer of Milwaukee, a place he had previously held from 1875 to 1878. While his prominence as a designer of water works overshadowed his sewerage engineering, he did some of the best work of his time in the latter line.

James P. Kirkwood, born in Scotland in 1807, was one of the most painstaking engineers connected with American sewerage work. He received his technical education as an apprentice to a Scotch engineering firm, and then came to the United States. From 1832 to 1855 he was engaged mainly on railroad work, in which he rose to high office, but was also occasionally employed by the federal government. In 1855, he undertook some difficult reconstruction of water mains in New York, which attracted so much attention that in the following year he was made chief engineer of the Brooklyn water works. Before this work was completed, his health became poor, and although he was subsequently consulted by many cities and planned many important water works he was unable to accept the numerous invitations to build the works he designed. His connection with sewerage plans was usually that of a court of final jurisdiction on the designs of others, and the conservatism of his views, as expressed in the old reports by him in the library of the American Society of Civil Engineers, is in contrast with those of the contemporary American advocates of extremely small pipes and other vagaries due to Chadwick and his followers in England. His most important original work in sewerage was probably in connection with an investigation of the pollution of Massachusetts rivers, made in 1876 for the State Board of Health. He was the second president of the American Society of Civil Engineers: he died in 1877.

Of all the engineers who were prominent in planning the earliest American sewerage systems, Col. Julius W. Adams is probably the best known today, for his treatise on "Sewers and Drains for Populous Districts," published in 1880, was widely used by engineers for at least 25 years, and his professional activities in many directions, such as talking the people of Brooklyn into starting the Brooklyn bridge, made him a well-known personage. His early engineering work was done on railroads, and it was not until 1857 that he undertook the sewerage of Brooklyn, mentioned in some detail in this Introduction. The book referred

Often they were constructed by individuals or the inhabitants of small districts, at their own expense and with little or no public supervision. In the early part of the nineteenth century water boards were not infrequently placed in charge of the sewerage works, which were usually mainly for drainage of storm water, as cesspools were generally employed for fecal matter. The last city to banish them was Baltimore; there were 80,000 of them in that city in 1879, according to a report of C. H. Latrobe, and many of them had overflow pipes discharging into the storm-water sewers, which was contrary to law. He estimated that the annual cost of cleaning these cesspools, at the contract price of $3 per load, was $96,000. As a result of the fouling of the soil by the contents of these pits, the City Health Commissioner reported in 1879 that of 71 samples of pump and spring water taken within the city limits, "33

to is very interesting as explaining the principles followed in the Brooklyn design, which proved to be too small in the larger sections, a fact he acknowledged without hesitation as soon as it was apparent and frequently mentioned as proof of the need of better knowledge of fundamental principles of design than he possessed in 1857. He was frequently retained later to pass on sewerage plans and wrote from time to time to the press on the subject, particularly while he was advisory editor of *Engineering News*. He was the sixth president of the American Society of Civil Engineers.

The Boston intercepting sewerage system was authorised by the legislature in 1876, on the basis of a report by E. S. Chesbrough, Moses Lane and Chas F. Folsom, the latter the energetic secretary of the Massachusetts State Board of Health. It was designed and partly built under the direction of Joseph P. Davis, who had gained experience under Kirkwood and Chesbrough, and was a successor of the latter as city engineer of Boston. His great modesty and deep aversion to a conspicuous position in public led him to decline on several occasions a nomination as president of the American Society of Civil Engineers. The intercepting sewerage system of Boston was the first great undertaking of the kind in this country, and gave its designer an international distinction as a sewerage specialist.

The sewerage system of Providence was declared in 1881 by Rudolph Hering, after a personal investigation of such work in our cities and in Europe, to be equal to anything abroad and much better than the work elsewhere in this country. The system was designed in 1869 by J. Herbert Shedd, then chief engineer of the water works and later city engineer, and its construction was under the personal supervision of his assistants, Howard A. Carson and Otis F. Clapp, later appointed city engineer. Mr. Shedd's report of 1874 on these sewerage works was long a famous engineering document. He designed his sewers to carry off 30 1/4 cu. ft. per minute per acre, without entirely filling their section, and employed a run-off formula providing for the effect of different slopes, with the result that his cross-sections proved large enough for their purpose. At the request of the mayor, the system was examined in 1876 by Gen George S. Greene, Col. J. W. Adams and E. S. Chesbrough, who reported that it was well designed and "the details of construction . . . have been carried out with a regard to important minutiæ which is rarely seen in such work." Owing to the later prominence of the Boston work, it is only right to point out that the Providence sewers formed for some years the model American system.

Edward S. Philbrick, born in Boston in 1827 and educated at Harvard, was engaged on railroad and structural engineering mainly down to the Civil War, when he became active in the work of the Sanitary Commission and thus had his attention turned toward public health matters. He was a great student of sewerage and sewage disposal problems and was occasionally engaged to report on them, but the greater part of his professional work remained in railroad and structural lines. His effect on American sewerage practice was a marked one, however, because he enjoyed writing about the subject for the press and talking about it before the students of the Massachusetts Institute of Technology and the municipal authorities of many towns and cities. Even after extensive business enterprises compelled him to give up active engineering practice, he continued to preach the gospel of good sewerage.

were filthy, 10 bad, 22 suspicious and 6 good." In 1906 Messrs. Hering, Gray and Stearns reported on a general plan for the sewerage and sewage disposal of this city, which led to the construction of a comprehensive separate sewerage system and disposal works.

There was a tendency in this country as elsewhere to construct the early sewers of needlessly large dimensions. One of the oldest sewers in Brooklyn was in Fulton Street. Although it drained an area of less than 20 acres and was on a grade of 1 in 36 it was 4 ft. high and 5 ft. wide. For many years the largest sewer in Manhattan was that in Canal Street, built somewhere between 1805 and 1810; it was 8 × 16 ft. in section and about 1850 was in very bad condition, being referred to by engineers of that time as affording instructive information of things it

It would not be proper to close this brief list without a mention of the unique position held by Dr. Rudolph Hering in the history of American sewerage. Like others named, he took up sewerage work by chance. He was engaged for a number of years in supervising the construction of various municipal works in Philadelphia and in this capacity he rebuilt some of the dilapidated structures of an earlier day, constructed in many cases with porous inverts for the purpose of admitting ground water and draining cellars. This led him to investigate the reasons for the failure of these old sewers, which proved such an interesting subject that he presented the matter as a paper before the 1878 annual convention of the American Society of Civil Engineers. It will be found in the Society's "Transactions," vol. vii, 252, and was not only the first, but also for many years the sole, American discussion of the design of sewer sections to carry the external loads coming on them. Although it was not so stated in the paper, the sections were designed to rest on platforms and resist the most unfavorable loadings to which such structures were exposed. The sections were thus somewhat heavier than would be needed under many conditions, but their publication was beneficial as counteracting a tendency at that time toward very light construction. This and other professional papers on allied subjects attracted attention to their author, and when the National Board of Health desired to make an investigation of European sewerage work, he was naturally selected, being a graduate of one of the best German polytechnic schools and familiar with American sanitary engineering practice. Bearing letters of introduction from a powerful semi-official body, he was able to gain the close acquaintance of the English and European sewerage engineers, and to ascertain what the leaders among them thought of the many disputed features of their work. His report of his work, forming the first clear American analysis of all the main problems of sewerage and the methods of solving them, established his reputation as specialist.

Finally, the name of D. E. McComb should be mentioned as the first American engineer who dared to build large sewers of concrete. Many wished to do this, but were afraid of the quality of the concrete which would be produced as a city job, just as this feeling of distrust lasted many years longer in Great Britian and led the Local Government Board to require in the case of reinforced concrete sewerage works an amortization fund corresponding to a life of 15 years only. Mr. McComb was superintendent of sewers in Washington and was convinced he could get good results. In 1883 Capt. R. L. Hoxie designed a 15 × 17½ ft. concrete sewer with a complete brick lining, which was built in 1885 under Mr. McComb's supervision; this sewer was 2500 ft. long and the maximum depth of trench was about 60 ft. Another concrete sewer designed and built at the same time had a circular section of 10 ft. diameter and a brick lining. These are the only concrete sewers in Washington with a brick lining in the invert and arch. In 1888, Mr. McComb constructed a concrete sewer 7.65 ft. in diameter and 864 ft. long, and in connection with it a gravel-catching basin of concrete, with an arch of 24.4 ft. and a rise of 4.5 ft., the thickness at the crown being 1.5 ft. Since that date the use of concrete in sewer construction has been the rule in Washington, the inverts being usually lined with vitrified brick. The success of the 1885 experiment led to the use of concrete for large sewers elsewhere, and it was soon demonstrated that they were less expensive than brick sewers and could be made without serious difficulties in securing good workmanship.

was wise to avoid. Its large size was doubtless made necessary by the existence of a brook at this place which was at one time provided with plank walls and was used by small boats, as illustrated in Valentine's "Manual of New York." In some cases the sewers were not only very large at their outlets but were continued of the same size to their heads; it was impossible to secure adequate velocity in such sewers unless they were laid on steep grades, and consequently some of them became offensive when the sludge accumulating in them underwent decomposition. In some cases the grades were in the wrong direction; an instance of this is mentioned in a report on Boston sewerage problems made in 1876 by E. S. Chesbrough, Moses Lane and Charles F. Folsom:

"The filling-in of the old mill pond naturally necessitated the extension of the sewers of that district to discharge into the canal; and, upon closure of the canal, the sewers were intercepted by a main which now discharges on both sides of the city, very irregular in grade, and whose two outlets are materially higher than its central point at Haymarket Square, thereby causing obstructions in that whole drainage district."

Such conditions as these produced the same nuisances which were so so marked in English and Continental cities in the middle of the last century. For instance, R. C. Bacot, superintendent of the Jersey City water and sewerage works, reported as late as 1865:

"The situation of these sewers and the necessity of their entire reconstruction has been brought to the notice of the proper authorities in my annual reports of the last four years, but nothing has been done by those immediately interested to remedy the evil. The outlet of the Henderson St. sewer (which is the receptacle of all these lateral sewers) being effectually closed up at the Morris Canal, no sewage matter can pass away, and consequently these sewers are almost entirely filled up with putrefying matter."

Much trouble was caused by the construction of sewers by private individuals and their subsequent acceptance by the city. As long ago as 1850, Rogers, Chesbrough and Parrott protested against such work in the following terms, in a report to the City of Boston:

"As the law now stands, any proprietor of land may lay out streets at such level as he may deem to be for his immediate interest, without municipal interference; and when they have been covered with houses and a large population are suffering the deplorable consequences of defective sewerage, the Board of Health is called upon to accept them and assume the responsibility of applying a remedy."

About the time that the last quotation was written there was considerable discussion among English engineers concerning the proper grades of sewers, and this controversy was duplicated on a less acrimonious plane in the United States. Lindley and Rawlinson were among the leading

advocates of flat grades with ample provision for flushing, while Wicksteed was probably the leading champion of enough slope to keep the sewers clean without other flushing than was afforded by the ordinary maximum daily flow. The low-grade school had its way with a vengence at Charleston, S. C., in 1857, where a sewer was built without any slope. It was 2-5/8 miles long, 3-1/2 ft. wide and 4-1/2 ft. high, with plank bottom and brick sides and arch. Each end had a tide gate, and the tides were such that a flushing current could be sent through the sewer at certain times in the day, strong enough to move broken brick, sand and clay.

Some of the difficulties which the American designer of sewers, without professional treatises of much value and lacking the help of the professional societies and journals of today, encountered in the middle of the last century are set forth in a report by Strickland Kneass, Chief Engineer of the Department of Sewerage of Philadelphia, in 1857:

"That portion of our charge which requires the most mature deliberation and careful examination is the arrangement of systems for drainage, with the proper proportioning of the sewers and drains constituting such systems, and has required a course of study and research that has been but little attended to in our city. It is a subject that has such a variety of elements within it as to have rendered it a matter of close investigation for a series of years in the city of London, by Comissioners appointed under acts of Parliament, the results of which are very voluminous and furnish much practical information, from which may be deduced laws of great value on the question of waterflow in sewers; yet so widely do they differ from experiments on record, made upon a small scale—upon which our mathematical formulas have been established—that judgment must be exercised in their adoption, but we hope to make such experiments upon some of the most perfect of our own sewers as will enable us to draw a comparison between their practical and theoretical value. Nevertheless, we have given the subject much consideration, and believe that the principles upon which we have arrived at the proportions of those sewers and drains already designed are correct, and will be found to be fully adequate to the purposes intended, yet with a strong hope that much saving may be made hereafter by a further reduction in the porportions of sewers for a given drainage."

The foul condition of the streets of Philadelphia at that time, owing to the filth discharged or cast into the gutters, is evident from another quotation from the same report:

"There should be a culvert on every street, and every house should be obliged to deliver into it, by underground channels, all ordure or refuse that is susceptible of being diluted. The great advantage in the introduction of lateral culverts is not only that underground drainage from adjacent houses should be generally adopted, but that by the construction of frequent inlets, our gutters would cease to be reservoirs of filth and garbage, breeding disease and contagion in our very midst."

About the time Kneass was hoping that experiments would enable him to adopt smaller sewer sections, another American city was undertaking the construction of a sewerage system, based on the best English data of that period, which taught a needed lesson of the danger of constructing sewers on any other basis than a complete understanding of the requirements of the locality they were to serve. The lack of such information was pointed out by the engineer of the works in question, the Brooklyn undertaking of 1857–9, which was designed by Col. Julius W. Adams, who later became the sixth president of the American Society of Civil Engineers. In his reports of that date he made these statements.

"The sewers in this city already built are too few in number, and their use too restricted and with too limited a supply of water, to enable us to derive from them data of any value whatever, and the attempt to obtain it by gaging the sewers in New York City, with the imperfect system which from past necessity has prevailed there, would be attended with a great expenditure of time, and from various causes, great uncertaintities would arise as to the value of the results obtained. No gagings, to our knowledge, have ever been made of sewers in this country, and very imperfect records exist of their dimensions, inclinations and other characteristics. If gagings have been taken, they have been too limited in scale to furnish data for a system of sewers in a city of so rapid a progression in population as Brooklyn promises to be; hence we are driven for the necessary information to those cities abroad where the subject has been forced on the public attention for a series of years.

"From recorded observations it appears that in a particular district, a rainfall of 1/2 in. in 3 hours took 12 hours before the flow in the sewer resumed its ordinary level on areas such as we are considering, and a rainfall of 1.1 in. in an hour and 0.3 in. in the next 2 hours occupied in discharging 15–3/4 hours; those points nearest the outfall draining off first, the most remote next, and some portions would be entirely clear before the water from the most remote points would reach the outfall.

"The present plan is calculated for a rainfall of 1 in. in an hour, to be discharged in 2 hours, or a discharge of 1/2 cu. ft. (3–1/4 gal.) per second per acre of area drained.

"It has been seen that we may estimate one-half of the flow of sewage, including all waste water due to 24 hours (everything but the rain) to run off in 8 hours, from 9 a. m., and that the sewage equals in amount 1–1/4 the water supply, or for 40,000,000 gal. water the sewage may be estimated at 50,000,000 gal., the half of which running off in 8 hours, gives 3,125,000 gal. of sewage per hour during 8 hours, which, from 12,000 acres, gives 260 gal. or 33 cu. ft. per acre per hour, or less than 0.01 in. in depth over the whole area, while the capacity of the sewer is calculated for an inch in depth."

To avoid intricacy of calculation and to err on the safe side by an excess in the dimensions of the pipes over the absolute requirements of the case, according to Colonel Adams' report, it was permissible to

employ for limited areas, at the summits of branch sewers, and elsewhere as experiment might dictate, the "formula for discharge from a still reservoir," but for larger areas and mains he preferred to be governed by Roe's gagings of the London sewers. The minimum inclination given to the sewers, when running half full, is stated in Table 2, and was considered great enough to produce a velocity "which will sweep away any substance which should be found in the sewers and many which should not. This quantity of water can be introduced at any time by the process of temporary dams or gates at the manholes, producing a sudden flush or scour of the sewer by water from the hydrants." This table is of interest in comparison with the authors' recommendations for minimum grades in Chapter III.

TABLE 2.—MINIMUM GRADES RECOMMENDED IN 1859 BY COL. J. W. ADAMS FOR SEWERS FLOWING HALF FULL

Diameter, in.	6	9	12	15	18	24
Slope, ratio	$\frac{1}{60}$	$\frac{1}{90}$	$\frac{1}{200}$	$\frac{1}{250}$	$\frac{1}{300}$	$\frac{1}{400}$
Slope, percentage	1.67	1.11	0.5	0.4	0.33	0.25

It might be added here that the recommendations for minimum slopes for brick sewers 36, 42 and 48 in. in diameter were 1 in 600, 700 and 800 respectively. By way of contrast reference may be made to the minimum grades adopted by C. Howard Ellers, Chief Eng. of Sewers of Chicago in 1881, which were 0.2 per cent., for 12- to 18-in. pipe and 0.1 per cent. for 20- to 30-in. sewers.

Although Colonel Adams was a leading student of sewerage problems and his plans were checked by James P. Kirkwood, a most careful and thorough engineer, the system proved inadequate, as is shown by the following quotation from a report of the chief engineer of the Brooklyn sewerage works on Dec. 23, 1870:

"Your engineer has been aware for several years of the importance of improving the sewerage system; and the frequent complaints of householders in certain localities of the city have caused the most careful investigations to be made from time to time." Many of the main sewers "proved to be too small since the districts have been built over, and are, in not a few instances, at too low a grade. The lower portions of many districts are frequently inundated, and what is proposed is a system of interception of the sewage and storm water of the upper portion of such districts; the lower sewers will then be ample in size to deal with the volume of flow which will be due to them."

The history of sewerage works has been marked until comparatively recent times by just such results of reliance on imperfect information for

design.[1] Much damage has been done by flooding cellars with storm water and sewage from surcharged sewers. Under the law of most states, which is explained in great detail in the famous New York case reported in 4 N. E. Rep. 321, if the city and the engineer follow out the legal requirements governing sewerage works, parties damaged by reason of defects due to mistakes in the design have no ground for action against the city. This shows the grave responsibility of the engineer and makes it incumbent upon him to utilize every possible resource whence information pertinent to the design may be secured. The legal rule in question was stated briefly as follows by the Maine Supreme Court in Keely *vs.* City of Portland, 61 At. Rep. 180:

> A municipal corporation is not responsible in damages for injuries caused to a person's property by the flowing back of water and sewage from a public sewer with which the property is connected, where this injury results from some fault in the location or plan of construction or in the general design of a sewer system, and not at all because of want of repair or failure of the municipality to maintain the sewer to the standard of efficiency of its original plan of construction.

A peculiar aspect of the subject was settled in 1905 by the Nebraska Supreme Court. In 1882 the city of Omaha adopted plans prepared by Colonel Waring for the sewerage of a part of the city, although the city engineer, Andrew Rosewater, protested against this action on the ground that the proposed lateral sewers were too small, being but 6 in. in diameter. The system was installed and it became necessary to build a larger sewer paralleling one of the laterals, except where it was on a steep grade. A property owner brought suit to enjoin the collection of special assessments for the larger sewer, contending that had the city followed the advice of its city engineer, it would have saved the money wasted on an inadequate system. The court ruled, however, that when "the city council, misled by the glamour of a great name, employed Colonel Waring, they did what any prudent, cautious business man would have done under like circumstances and the plaintiff cannot complain if their judgment was erroneous."

[1] Among the unique sewerage systems built in early days in the United States that in the older part of San Francisco has an exceptionally prominent place, for the methods of design and construction were marked by a complete disregard of proper engineering principles, as is evident from the following quotation from a paper by C. E. Grunsky on "The Sewer System of San Francisco" in Trans. Am. Soc. C. E., lxv, 294:

"The plan . . . seems to have been to construct egg-shaped brick sewers, 5 ft. high and 3 ft. wide, in all streets and alleys where property was valuable and could afford to pay for large sewers. . . . The size of sewer was frequently determined by the Superintendent of Streets, who was never a civil engineer. . . . The invert, as required by ordinance, was placed 10 ft. below street grade, generally level, or, due to the intelligence of most of the sewer contractors, a few inches lower at the down-hill side of the street intersection. The sewers in the intersection might connect with other brick sewers of like size, or with larger sewers, or with small pipe sewers, according to what was prescribed at some other time, for the streets leading from the intersection."

The sufficiency for its purpose of one of the largest sewers in the country was approved by the Missouri Supreme Court in the case of Gulath *vs.* City of St. Louis, 77 S.W. Rep. 744. This related to the Mill Creek sewer in that city, draining about 6,400 acres and begun in 1864. At its upper end it is 10 ft. in diameter and at its lower end, 5 miles distant, it is 16 ft × 20 ft. in section. It was designed to care for a rainfall of 1 in. per hour. Before it was built the site of the plaintiff's store was overflowed by the creek many times, according to testimony. After the sewer was constructed, the site was overflowed but three times down to the date of the suit, and on each occasion after an unusual storm. The court ruled that such exceptional storms need not be taken into account by the engineer in designing such works.[1]

Although where a properly authorized official or committee adopts plans for a sewerage system it cannot be held responsible in most states for damages resulting from defects of design, it has been held by some courts, as the Wisconsin Supreme Court in Hart *vs.* City of Neillsville, 104 N.W. Rep. 699, that the mere existence of sewers will not be considered the equivalent of a plan. In that case the court held that if a sewerage system was constructed without a properly adopted plan, the city is liable for any damages that may result from defects in it. The court also ruled that though a city was not liable for damages to private property caused by mere defects in a properly adopted and executed plan, if it was informed of such defects and the direct continuing injury to private property that would result unless they were remedied, it should exercise ordinary care to prevent such a result and was responsible for damages caused by any negligence in this respect. This ruling indicates that when a city takes over the improvements made upon a large tract of land, such as the "additions" so frequently absorbed where communities are developing rapidly, the plan and construction of the sewerage systems should be very carefully scrutinized before the papers are finally signed.

In the design of sewerage systems down to a comparatively recent date there seemed to be a strong preference for outfalls in tidal waters

[1] This was expressed in the following words in a preliminary report by the New York Metropolitan Sewerage Commission: "The importance of giving careful consideration to the rainfall is greater in designing collecting systems of sewerage than in providing for final disposition. The function of such sewers is not only to carry off the drainage of the houses, but to prevent accumulations of water in the streets. It sometimes happens, when excessive falls of rain occur, that sewers are surcharged. At such times the drainage of houses is interfered with and often stopped, in which case cellars may be flooded and other serious inconvenience produced. It is usually impracticable to provide combined sewers of a size and grade sufficient to carry away the water which falls in storms with sufficient promptness to insure that inconvenience from flooding shall never occur. At long intervals rainfalls of exceptional severity take place, and to provide for these sewers would have to be built so very large that they would represent a considerable investment over the sum required to give them sufficient capacity for all the ordinary and most of the heavy rains which are likely to fall."

which were locked by flood tide, and it was by no means rare to find the outlets at an elevation which insured their submergence at mean tide. In its investigations of the sewerage systems discharging into New York Bay the Metropolitan Sewerage Commission reached the conclusion that two opinions led to this construction, the first that the sewer bottom should be given as much slope as possible in the belief that it controlled the velocity of flow in the sewers and the other that the wind blowing into the open ends of the sewers drove the foul air up into the streets through the perforations in the manhole covers.[1]

Another cause of flooding existed in some sewerage systems otherwise free from defects. This was the preparation of sewer plans by using the invert grade or bottom slope, for calculating capacities, instead of the hydraulic grades or slopes of the water surface in the sewers. The result of this mistaken policy in Brooklyn down to 1907, was "to produce sewers that would overflow at manholes and be, so to speak, drowned out whenever the flow approximated the maximum capacity." (Report Metropolitan Sewerage Commission, 1910).

The United States suffered, just as England did at an earlier date, from the improper design of separate systems of sewerage in which the house sewage and rain water are kept nearly or quite distinct. Just who designed the first system of sewers for removing house sewage separately is not definitely known, but the principle was strongly advocated as early as 1842 by Edwin Chadwick. He has been called the "father of sanitation in England," and unquestionably played an important rôle in arousing that country to the need of greater cleanliness not only in cities but also in rural districts. He was a man of convincing address, great self-reliance and enthusiasm, and strong imagination which was unfortunately not restrained by technical knowledge. As a result he advocated, even in meetings of engineers, so-called hydraulic principles and some features of design that were wholly incorrect and at last resulted in his being publicly branded as a charlatan at a meeting of the Institution of Civil Engineers at which he was in attendance (see Proc. Inst. C. E., xxiv).

[1] That this erroneous practice had been abandoned by leading engineers before the birth of many of the readers of these pages, attention is called to the following statement in a report on the sewerage of Brookline, Mass., made in 1875 by E. S. Chesbrough, W. H. Bradley and Edward S Philbrick: "With regard to the height of the outfall, two important reasons exist for keeping it as high as possible: viz., to prevent the influx of tide water at the mouth, and to afford an advantageous connection with any intercepting sewer which may hereafter be constructed on the south side of the Charles River for Boston and vicinity. On the other hand it is extremely desirable to keep the outlet as low as possible, both to secure an efficient inclination to the sewer and to drain as well as may be the low-lying district. . . . We therefore recommend that the bottom of the outfall be placed at the level of half-tide, and that a self-acting tide gate be placed there. Should a grand scheme ever be carried out for marginal intercepting sewers for Boston, it is probable that resort must be had to pumping to make such a scheme successful, in which case the low level above named for the outlet of the Brookline sewer will not be found objectionable."

The principle of the separation of house sewage from rain water, advo-ated by Chadwick,[1] was so meritorious for many places that it was eveloped along rational lines by a number of leading English engineers, otably Sir Robert Rawlinson, whose "Suggestions as to Plans for Main ewerage, Drainage and Water Supply," published by the Local overnment Board, did much to prevent the laying of sewers of too small ze and poor alignment, without proper facilities for the cleaning which likely to be necessary in all such works.

The separate system received much study by American engineers, s was natural in view of their reliance on English practice for precedent. 'ortunately, however, the difference between the character of the rainfall England and the United States was known here and its influence on he design of sewerage works was appreciated. The English rains are nore frequent but less intense, and hence our storm-water drains must e larger for like topographical conditions. Our heavier rains afford nore vigorous flushing action in the sewers, so that the necessity for the ather elaborate provisions for flushing combined sewers in many Euro-ean cities is not so evident here. Wherever the surface drainage could e cared for satisfactorily at a low cost without the use of large combined ewers receiving both house sewage and rain-water, there was a manifest dvantage in adopting the separate system, which was used at about the ame time in designs prepared by Benezette Williams for Pullman, Ill., nd George E. Waring, Jr., for Memphis. The Memphis system was he most conspicuous, although a comparative failure, a fact which the eople of the city naturally suppressed for business reasons for many 'ears. Colonel Waring received two patents, Nos. 236740 and 278839, ssued in 1881 and 1883, for separate sewerage systems, and his use of hese patents in ways which many engineers regarded as unprofessional rought severe criticism upon him.

During the summer of 1873 more than 2000 persons died of yellow ever in Memphis. In 1878, 5150 deaths occurred from the same ause; a rigid quarantine and sanitary regulations were enforced but he disease was merely checked and during the next year was the cause f 485 deaths. The Legislature authorized unusual taxing and adminis-rative methods in the stricken city, whose affliction aroused the sym-athy of the whole nation and was largely responsible for the formation f the National Board of Health. A committee of the Board sent Colo-

[1] John Phillips, in a paper read before the Philosophical Society of Glasgow, Feb. 7, 1872, aid; "The principle of drainage in towns which I advocate, and which was first proposed y me, is called the Separate System. (It is generally thought Mr. Menzies is the originator f this system, but this is not the fact.) I had matured and proposed it nearly 8 years efore he resuscitated it in 1855. This was in 1847, when I was Chief Surveyor of a large ortion of the Metropolitan (London) sewers. . . . In my preliminary report in 1849 n the drainage of the Metropolis (London) I proposed this system for adoption. But ublic opinion was not then prepared for this advanced idea, and, in consequence, my roposal not only met with no support, but with considerable opposition."

nel Waring to the city, which was inspected and surveyed under his supervision. The maximum sum that could be raised by taxation for sewers was $368,702, and sewerage was greatly needed so it was necessary to make the money go as far as possible.

Colonel Waring designed a separate system using 6-in. lateral sewers with a 112-gal. flush-tank at the head of each, discharging once in 24 hours. The house drains were 4 in. in diameter. Not more than 300 houses were to be connected with a 6-in. sewer; if there were a larger number to be provided for the pipe was to be enlarged to 8-in. toward its lower end. The main sewers were made of 10-, 12-, 15- and 20-in. pipe; all of them were underdrained. All rain-water was supposed to be excluded and the sewers were ventilated through the soil pipes in the houses. There were no manholes at first and the lampholes for inspecting the interior of the sewers were a failure from the outset, because the vertical shaft was heavy enough to crush the small pipe from which it rose. In 1880, 24.2 miles of sewers were built under Colonel Waring's direction and 2.1 miles of old sewers were bought, the 26.3 miles costing $183,086. During the next 2 years 12.3 miles were built and bought, and in that period there were 75 obstructions of the 4- and 6-in. sewers, costing $1112 to remedy. The main lines in some places were reported by the City Engineer, Niles Meriwether, to be taxed to their full capacity. In 1883–4, 2.3 miles were added to the system and 164 obstructions were removed at a cost of $1982. During 1885–86, 2.58 miles of sewers were constructed and $2172 spent for removing obstructions. The inadequate capacity of the larger sewers had resulted in the construction of a relief sewer during this period. By that time engineers familiar with the conditions were convinced that some of Colonel Waring's favorite details had proved defective, and that the Rawlinson type of separate system, with larger pipes laid without vertical or horizontal bend between successive manholes, was preferable. The partial failure of the so-called Waring system was demonstrated, therefore, in about 5 years' experience at Memphis; this was a little longer than was required to demonstrate the same thing at Croydon, England, 30 years before the Memphis experiment. The Croydon system was made up of 6350 ft. of 4-in. sewers, 44,436 ft. of 6-in., 6435 ft. of 8-in., 14,100 ft. of 9-in., 2469 ft. of 10-in., 3324 ft. of 11-in., 12,117 ft. of 12-in., 9518 ft. of 15-in., 1506 ft. of 18-in. and 36 ft. of 21-in. In a period of 20 months in 1852–53, there were 60 stoppages in the 4-in. sewers and 34 in the 6-in., but not more than one in any of the other sizes.

There was so much personal magnetism in Colonel Waring that he was able to use the prestige of his sanitary achievements at Memphis to impress his views regarding small pipe sewers on a number of communities. The National Board of Health felt some distrust regarding such systems soon after its formation, and it accordingly sent Rudolph

Hering to Europe on a tour of investigation, which lasted nearly a year. On his return he prepared the elaborate report on the principles of sewerage and their exemplification in the best works of Europe already referred to, which remains to this day a thorough summing up of good practice. It is not often that an engineering monograph retains its value for more than a quarter of a century. As a result of his investigation Dr. Hering outlined the respective fields of the separate and combined systems as follows:

"The advantages of the combined system over a separate one depend mainly on the following conditions: Where rain-water must be carried off underground from extensive districts, and when new sewers must be built for the purpose, it will generally be cheaper. Its cost will also be favorable in densely-inhabited districts from the circumstances that the proportion of sewage to rain-water will be greater, and therefore increase the sizes of the separate sewer pipes, yet without decreasing those of the rain-water sewers; while the sizes of the combined would not vary with the population, because the quantity of sewage is less than the quantity within which the amount of storm-water can be estimated. But more important is the fact that in closely built-up sections, the surface washings from light rains would carry an amount of decomposable matter into the rain-water sewers, which, when it lodges as the flow ceases, will cause a much greater storage of filth than in well-designed combined sewers which have a continuous flow and generally, also, appliances for flushing.

"The separate system is suitable——

"Where rain-water does not require extensive underground removal and can be concentrated in a few channels slightly below the surface, or where it can safely be made to flow off entirely on the surface. Such conditions are found in rural districts where the population is scattered, on small or at least short drainage areas, and on steep slopes or side hills.

"Where an existing system of old sewers, which cannot be made available for the proper conveyance of sewage, can yet be used for storm-water removal.

"Where purification is expensive, and where the river or creek is so small that even diluted sewage from storm-water overflows would be objectionable, especially when the water is to be used for domestic purposes at no great distance below the town.

"When pumping of the sewage is found too expensive to admit of the increased quantity from intercepting sewers during rains, which can occur in very low and flat districts.

"Where it is necessary to build a system of sewers for house drainage with the least cost and delay, and the underground rain-water removal, if at all necessary, can be postponed.

"The principle of separation, although often ostensibly preferred on sanitary grounds, does not necessarily give the system in this respect any decided advantage over the combined, except under certain definite conditions. Under all others, preference will depend on the cost of both construction and maintenance, which only a careful estimate, based on the local requirements, can determine."

The cost of sewerage works is a subject presenting many pitfalls to those without experience, and even to those having it. The fluctuations in the rate of wages and the price of materials from year to year, the character of the workmanship required and of the supervision by inspectors, the competence of the superintendents of construction and the introduction of labor-saving machinery, these and other factors which affect the cost of public works are not readily explained quantitatively, so that a public official or young engineer can grasp their combined effect. This effect is marked, however, as is well shown in Fig. 1, from the 1910 report of E. S. Rankin, engineer of sewers of Newark, N. J. This diagram shows the fluctuation in the contract price of 12-in. pipe sewers in 8 to 10-ft. trenches, during a period of 25 years. The costs plotted in the diagram were those of contracts for work of practically the same character and show a range from 51 cents to $1.15. Records of this character can be duplicated in most cities where costs have

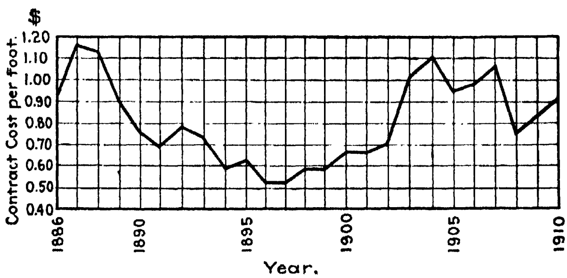


FIG. 1.—Contract costs of 12-inch pipe sewers in Newark N,. J., during a period of 25 years.

been carefully kept for a long time and they prove how wary an engineer must be in preparing estimates.

The disposal of the sewage of most cities, until recent years, was carried out in the easiest way to become rid of it, without much regard to unpleasant conditions produced at the place of disposal. Irrigation with sewage was apparently practised at ancient Athens, but there is very little definite information of any methods of disposal on land down to about 300 years ago, when sewage farming was successfully introduced at Bunzlau, Germany. The earliest municipal work of the kind in Great Britain was on the Craigentinny meadows of about 400 acres extent receiving the sewage of a part of Edinburgh for about a century. The subject of disposal received only occasional local attention, however, until the construction of sewerage systems after the cholera epidemics of 1832–33 and 1848–49. Owing to the small size of British streams, their

pollution by the sewage discharged into them soon became a nuisance. Interference with agricultural and manufacturing uses of water was apparently at first given more attention than any danger to health. When the cholera epidemic of 1854 had been suppressed, Parliament passed the comprehensive nuisances removal act of 1855, to which reference has already been made. This did not make sewage treatment compulsory, however, nor did the rivers pollution prevention act of 1876, although as early as 1865 a royal commission had reported:

"First, that whenever rivers are polluted by a discharge of town sewage into them, the towns may reasonably be required to desist from causing that public nuisance.

"Second, that where town populations are injured or endangered in health by a retention of cesspool matter among them, these towns may reasonably be required to provide a system of sewers for its removal."

Two methods of treating sewage came into vogue about the time of this report. The irrigation of land by sewage was the older of these but the precipitation of the solids and some of the dissolved matter by chemical treatment and subsequent sedimentation attracted more attention owing to its exploitation by promoters as well as to the favorable opinion of it held by many careful and conservative engineers. A special committee appointed by the Local Government Board in 1875 reported on the whole subject as follows:

"That most rivers and streams are polluted by a discharge into them of crude sewage, which practice is highly objectionable.

"That, as far as we have been able to ascertain, none of the existing modes of treatment of town sewage by deposition and by chemicals in tanks appears to effect much change beyond the separation of the solids and the clarification of the liquid. That the treatment of the sewage in this manner, however, effects a considerable improvement, and, when carried to its greatest perfection, may in some cases be accepted.

"That town sewage can best and most cheaply be disposed of and purified by the process of land irrigation for agricultural purposes, where local conditions are favorable to its application, but that the chemical value of the sewage is greatly reduced to the farmer by the fact that it must be disposed of day by day throughout the entire year, and that its volume is generally greatest when it is of the least service to the land.

"That land irrigation is not practicable in all cases; and, therefore, other modes of dealing with sewage must be allowed.

"That towns, situated on the sea-coast or on tidal estuaries, may be allowed to turn sewage into the sea or estuary, below the line of low-water, provided no nuisance is caused; and that such mode of getting rid of sewage may be allowed and justified on the score of economy."

The density of population in England and the very small amount of land well suited for sewage farming and filtration led to particular interest in intensive methods of treatment, whereby in plants of compara-

tively small area the sewage was rendered suitable for a final treatment on land, which was practically compulsory for most English systems discharging into fresh water. This constraint was exercised by the Local Government Board, without whose approval money could not be raised for public works except by special act of Parliament; the Board was wedded to a final land treatment until comparatively recently. Consequently septic tanks, trickling filters and contact beds were received with acclamation and tested on a practical scale that was unwarranted, for instance, in Germany.

The disposal of sewage in the United States did not receive so much attention 30 years ago as in England, nor does it yet, because the extent of the nuisance caused by its discharge into water was not so marked and because of the greater area of land suitable for broad irrigation or intermittent filtration on beds graded *in situ* and of relatively cheap materials suitable for the construction of artificial disposal beds. Its importance was foreseen by the Massachusetts State Board of Health early in the seventies, and its secretary, Dr. C. F. Folsom, made a careful study of disposal in Europe, which resulted in 1876 in a report which was the most complete statement that had been made of the state of the art at that time. Irrigation and filtration were introduced in a few places, but it was not until certain rivers in Massachusetts became quite offensive that any work on a large scale was undertaken. The first extensivé treatment plant utilized chemical precipitation and was built at Worcester, Mass., in 1889–90, from the plans of Charles A. Allen with the advice of James Mansergh of London and Prof. Leonard P. Kinnicutt of Worcester. It was intended to abate the nuisance caused by the discharge of crude sewage into the Blackstone River, which it accomplished, and it has furnished a large amount of practical information regarding various methods of sewage treatment, for elaborate experimental work, some of it on a very large scale, has been encouraged by the city authorities in the belief that the small expense of such research would be well repaid by the use that could be made by the municipality of the results in planning extensions of the original installation and improving the methods of treatment. About the same time, the Massachusetts State Board of Health, which had been given large powers of control over the disposal of sewage, established the Lawrence Experiment Station for the study of both water and sewage treatment; the influence of the research work done there has been deep and far-reaching, being particularly noteworthy for the prominence given in early years to intermittent filtration, a method of disposal neglected in England on account of the limited tracts of land suitable for practising it.[1]

[1] Although not used widely, this method of disposal had been employed for a number of years and was described in detail in Bailey-Denton's "Ten Years of Intermittent Downward Filtration," which was published in 1881.

While these introductory notes are intended merely to show how the principles of sewerage and sewage disposal became established on a firm footing in engineering practice and not to review the development of the details of the subject, particularly of late, it should be stated here that recent progress has been wonderfully rapid. When the reason for this is sought, it will be found in that admirable spirit of good-will and co-operation existing among American engineers, which not only finds expression in the work of the engineering societies but also in the close and friendly contact maintained by engineers in this country with one another and with the engineers of other countries. This has been a good influence on American sanitary engineering, for it has led to friendly personal relations, open minds and a recognition of the work of others by giving credit where credit is due, which have combined to concentrate attention on those subjects where progress was most needed and to prevent the needless duplication of effort in striving for the same goal. So long as this spirit persists. American sewerage engineering will go forward buoyantly.

Disposal by dilution has retained greater favor in the United States than in England because of the larger bodies of water available for receiving the sewage. The first comprehensive American study of the subject was begun in 1887 by Dr. Hering for Chicago, and resulted in his recommendation of a drainage canal to dilute the sewage with water from Lake Michigan and deliver it into the Desplaines River, flowing into the Illinois, a tributary of the Mississippi. Since then many other studies have demonstrated that, so far as the prevention of nuisance is concerned, disposal by dilution is the most economical method of becoming rid of sewage at many cities.

Dilution is now (1914) under fire, however, from some health officers and their engineers, who oppose the discharge of merely screened and settled sewage into rivers or lakes furnishing water for potable purposes. While there is substantial agreement that it is less expensive to obtain good water by filtering a sewage-contaminated supply than to treat the sewage so elaborately that there is no danger attending the discharge of the effluent into this supply, it is claimed by some sanitarians that it is unsafe to rely exclusively upon the continuous proper operation of water filters and the treatment of sewage is also necessary to protect the public health. The subject is one of the most disputed features of sewerage today; it is destined to concern many cities vitally and to involve them in enormous financial obligations if the advocates of compulsory sewage treatment have their way. The sanitary engineer who neglects to work for the best interests of the public health falls short of the full discharge of his professional obligations, but it is wise to keep in mind a fact stated as follows by *Engineering News:* "We know of many instances in which business men distrust engineers and pin their faith to so-called 'practical'

men, largely because of unfortunate experience with engineers who appeared to think that the question of cost was no part of their concern."

The legal dangers of attempting to discharge sewage into a small body of water must be considered in the design of sewerage systems. In Sammons *vs.* City of Gloversville, the New York Court of Appeals decided that although the city exercised a legitimate governmental power for public benefit when it built its sewers, it had no charter rights to discharge sewage into a brook in such a way as to injure the plaintiff's lands below the point of discharge. Even where a city has statute rights to construct sewers emptying into a creek, whereby a nuisance was created, the Alabama Supreme Court held in Mayor, etc., of Birmingham *vs.* Land, 34 S. Rep. 613, that the owner of a riparian farm below the sewer outlet was entitled to damages. The Maryland Court of Appeals similarly decided the case of West Arlington Imp. Co. *vs.* Mount Hope Retreat, 54 Atl. Rep. 982. The fact that a water-course is already contaminated does not entitle other persons to aid in its contamination or prevent those thereby injured from recovering from them damages for the injury; Ind. Sup. Ct., West Muncie Strawboard Co. *vs.* Slack, 72 N. E. Rep. 879.

The case of Waterbury, Conn., was of much interest for many years because of the protracted fight made by the city against building purification works in accordance with a decree of the Connecticut Supreme Court going into effect on Dec. 1, 1902. In one of the subsequent decisions in this litigation, the court stated that the construction of the Waterbury sewers in 1884, in accordance with the terms of its charter, was lawful and that their construction to discharge sewage into the Naugatuck River gave nobody cause of action. The sewers could be used for that purpose without any invasion of the rights of owners of riparian property below the point of discharge. But when the city discharged sewage into the river in such quantities and in such manner that it was carried without much change to the property of a manufacturing company, thereby producing a public nuisance to the company's special damage, the city was held to make a public nuisance of its sewerage system. Each day such an unlawful act was repeated the company suffered a fresh invasion of its legal rights, according to the court.

CHAPTER I

THE GENERAL ARRANGEMENT OF SEWERAGE SYSTEMS

It has been pointed out in the introductory chapter that many of the troubles with early sewerage systems were due to an underestimate of the amount of the rainfall reaching the sewers and an overestimate of their capacity. At a later period another error of judgment was often made, which is causing trouble now; this was the failure to plan works capable of extension on the original lines after the cities had grown much larger. There is a limit, of course, beyond which an engineer is not justified in making allowances for the requirements of the future, but the former neglect to look ahead for more than a relatively few years has recently made very expensive works necessary in a number of cities. It is not wise to place a heavy financial burden on the present generation for the benefit of those to come, but if future expenses can be reduced by careful planning today, without appreciable additional cost, such a course is manifestly the right one to adopt.

One cause of the confusion that sometimes arises in considering sewerage plans, is a failure to recognize that there are distinct general arrangements of sewers and there are several distinct classes of sewers, each having a main purpose.

CONDITIONS GOVERNING A SEWER PLAN

The general outline of a sewerage system is governed by two prime factors, the topography of the city and the place of disposal of the sewage. The two are sometimes so simple in their effect that the general plan to be followed is self-evident, but in other cases they have complex interrelations that require protracted study before the best plan can be definitely determined.[1]

Influence of Disposal Methods.—There are three general methods of disposal that affect the design of the sewers.

The first method of disposal is directly into a river or other body of water on the shore of which the city lies; probably the Borough of Manhattan offers the best example of this, with its main sewers running east and west to numerous outlets on the North and East Rivers.

The second method of disposal is to intercept the sewage and carry it

[1] The first general discussion of this subject in English was apparently in Hering's 1881 report to the National Board of Health.

to a point in the adjoining body of water where it will not cause trouble; this may not be necessary at first, but in most cases it is inevitable if the city grows as rapidly as do most American municipalities, and attention must be paid to it, particularly to the future desirability of separating the house sewage from part of the storm water. The Cleveland system, shown in Fig. 2, from *Engineering News*, March 28, 1912, is an example of this intercepting plan.

The third method of disposal is by some treatment of the sewage

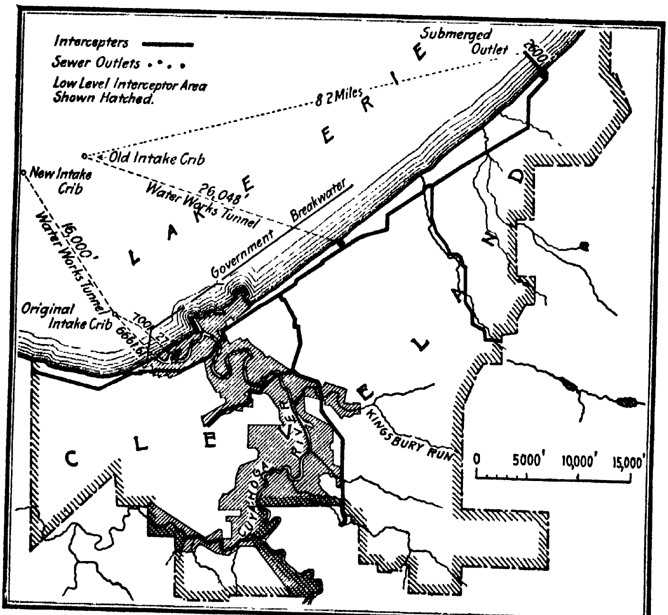


Fig. 2.—Cleveland intercepting sewer system.

which will materially change its character before it is discharged into a body of water. This makes it necessary to deliver the sewage to treatment works, suitable sites for which are difficult to procure in many cases, particularly where the country is well built up, not enough open land properly located is available in the city, and neighboring towns object to the plant being located within their limits. The separation of storm water from the house sewage often becomes financially advisable, so as to permit the former to be discharged by short, direct lines into the river, lake or bay nearby, and also keep down the cost of the long sewer

to the disposal works, and the disposal costs as well. In the case of combined sewers, the same end is attained by making provision at one or more points for the discharge of the storm water in excess of a predetermined amount, through overflow weirs or chambers into channels or other outlets leading directly to the river or lake. The early flow of storm water carries a large amount of organic matter from the streets into the sewers and takes into suspension some of the matter deposited previously in the sewers, and its treatment is often considered as desirable as that of the house sewage.[1]

The design of the overflow chambers is thus an important matter. It may be found practicable to permit a large proportion of the sewage to escape through some of them in the early years of their use, but later, owing to a change in the character of the body of water receiving this excess storm-flow, or the greater impurity it may then possess, its delivery to the disposal works may prove desirable. While it is unnecessary in many cases to give the outfall sewer to the works a very large capacity to provide for such future possibilities, owing to the heavy fixed charges such construction will cause, it is often desirable to consider future requirements with particular care in planning the overflow chambers, in order that their reconstruction or modification may not cause difficulties in the operation of the system out of all proportion to the cost of the work.

Influence of Topography.—The topographical features of a city also have a marked influence on the design of a sewerage system. In a large city situated on a flat plain without any neighboring river or lake into which the sewage may be discharged without elaborate treatment the radial system may prove best. This has its most elaborate development in Berlin, where it was introduced by Hobrecht. The city is divided

[1] The Local Government Board of England generally required until recently that any increase in the flow in combined sewers up to three times the normal dry-weather rate should be treated like house sewage, and that six additional dilutions should be passed through "storm-filters" of gravel, broken stone or clinker. In the fifth (1908) report of the Royal Commission on Sewage Disposal, these requirements are criticized thus: "These requirements should, we think, be modified; they are, in our opinion, not sufficiently elastic, and experience has shown that special storm-filters, which are kept as stand-by filters, are not efficient. We find that the injury done to rivers by the discharge into them, of large volumes of storm-sewage chiefly arises from the excessive amount of suspended solids which such sewage contains, and that these solids can be very rapidly removed by settlement. We therefore recommend, as a general rule, that, (1) Special stand-by tanks, two or more, should be provided at the works and kept empty for the purpose of receiving the excess of storm-water which cannot properly be passed through the ordinary tanks. As regards the amount which may be properly passed through the ordinary tanks, experience shows that in storm times the rate of flow through these tanks may usually be increased up to about three times the normal dry-weather rate, without serious disadvantage. (2) Any overflow at the works should only be made from these special tanks, and this overflow should be arranged so that it will not come into operation until the tanks are full. (3) No special storm-filters should be provided, but the ordinary filters should be enlarged to the extent necessary to provide for the filtration of the whole of the sewage which, according to the circumstances of the particular place, requires treatment by filter."

into a number of sectors and the sewage of each sector is carried outward by pumping to its independent disposal farm, or the trunk sewers of two or more sectors may be connected to a farm. There were eight farms in 1910. The advantage of this system is that most of the sewers are likely to be of adequate capacity for a long period, and the large, expensive sewers are reduced to their minimum length.

"The sewage of each district is pumped through force mains to the irrigation farms, which, with such an arrangement, can be divided around the suburbs of the city. The water courses and, in part, the low ridges, form the limits of the districts, whose number has now risen to 12, and whose size varies between 672 and 2128 acres. The pumping station is located at the lowest level possible, and in only one district is an intermediate pumping station necessary. The advantages resulting from this arrangement are so great that the increased cost of pumping due to the division of the pumping capacity is unimportant and can be counterbalanced by the greater security of operation. The overflow works, for which the water-courses of the city act as outlet channels, form an important feature of the system" (Frühling, "Die Entwässerung der Städte," 1910).

In most cases such an arrangement is rendered impracticable by the existence of hills, water-courses and other topographical conditions. Usually, moreover, old sewers complicate the problem, for it is always desirable to utilize existing structures so far as practicable. Only in rare cases does the engineer have an opportunity to design a complete sewerage system for a large city, as was the case in New Orleans and Baltimore.

In Baltimore, where the sewage had to be taken 5-3/4 miles outside the city for treatment, it was apparent that the storm water should be collected separately, for there was no objection to its discharge into the nearest water-courses adapted to receiving it. The city is intersected by four streams, which discharge into branches of the Patapsco River. One of these streams receives so much foul run-off that it has been covered over; the others are open. The Patapsco and its branches are tidal arms of Chesapeake Bay. The drainage area was divided into 28 districts, and the storm-water drains in each one were planned independently of the rest, to fit the topography and arrangement of streets in the best way. Those drains were kept as close to the surface as possible, in order not to force the sewers so low that it would be difficult to connect the houses with them. In one low-lying district where the drainage problem was particularly difficult, the plans called for raising the street grade and building a drain to carry the storm water into the Patapsco River instead of a nearer stream which was liable to have its surface raised considerably during floods, a condition which might cause a surcharge of the drains emptying into it.

The removal of the house sewage was a much more complicated prob-

lem. Part of it comes from districts which are high enough to enable the sewage to flow by gravity to the treatment works, but a large part has to be pumped. The contour line between these two service districts was determined by two factors, the elevation at which the sewage must be discharged at the treatment works and the minimum safe grade of the outfall sewer from the city to the works. The accompanying plan of the intercepting sewers, Fig. 3, from *Engineering Record*, Dec. 5, 1908, shows where the outfall sewer reaches the eastern boundary of the city and is continued through it toward the western boundary as a high-level intercepter, receiving all the sewage that can be delivered by gravity to the disposal works. The sewage of the low-lying portions of the city is collected by four intercepting sewers, two of which contain small

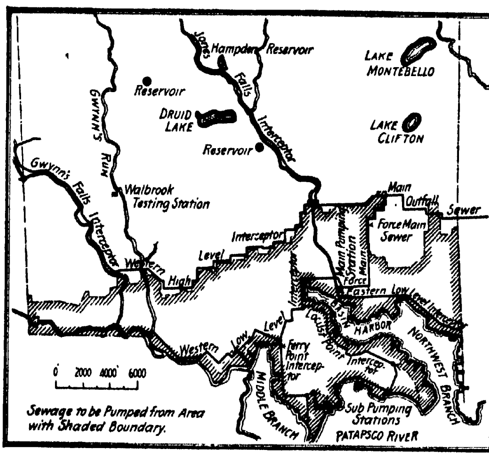


FIG. 3.—Baltimore intercepting sewer system.

pumping plants to lift the sewage enough to prevent a deep position of these sewers, which is undesirable on account of the high cost of construction in deep trenches in water-bearing soil, and the difficulty of connecting the tributary sewers satisfactorily with deep-lying intercepters. All these intercepters run to a station containing five pumps, each with a nominal rating of 27,500,000 gal. a day against a total head of 72 ft. These pumps force the sewage through two lines of 42-in. cast-iron mains 4550 ft. long into a sewer about a mile long, discharging by gravity into the main outfall sewer.

Between the arrangement of sewers in the Borough of Manhattan, discharging both storm water and house sewage through short lines into the nearby rivers, through many outlets, and the arrangement at

Baltimore, with its separation of the storm water and the sewage, its high and low levels, pumping stations, long outfall sewer and elaborate sewage treatment works, there is an infinite variety of combinations practicable. In every case, however, the topography suggests the natural drainage and the street plan exercises a more or less strong modifying influence. One of the most experienced old-school American engineers, William E. Worthen, the seventeenth president of the American Society of Civil Engineers, when he was retained to plan important sewerage improvements in Brooklyn, had constructed a large relief map of the district, in order that he might see the whole topography of the area clearly while considering the existing troubles and the various remedies for them. While such a map is unnecessary in most cases, of course, topography is sometimes far more important than street plans. In every case special attention should be paid to the low-lying districts, for it is there that the largest sewers must be built in many cases, and the difficulties of construction are the greatest. It may be found advisable to reduce such work to a minimum by constructing an intercepting sewer at a somewhat higher level and thus restrict the construction in the low-lying sections to small sewers only deep enough to serve the property of that district.

Another influence of topography on sewerage plans, often overlooked, was stated as follows by Dr. Hering in his report of 1881 to the National Board of Health:

"In case of sudden showers on a greatly inclined surface which changes to a level below, the sewers on the latter will become unduly charged, because a greater percentage flows off from a steeper slope in a certain time. To avoid this uneven reception, the alignment should, as much as possible, be so arranged as to prevent heavy grades on the sloping surface, at the expense of light ones on the levels. In other words, the velocity should be equalized as much as possible in the two districts. This will retain the water on the slopes and increase its discharge from the flat grounds, thus corresponding more to the conditions implied by the ordinary way of calculating the capacity of sewers. It will therefore become necessary not to select the shortest line to the low ground, but, like a railroad descending a hill, a longer distance to be governed by the gradient. This does not necessarily imply a longer length of sewers for the town, because more than one sewer for a street is not required by it."

Still another decided influence of topography is shown where the configuration and surroundings of the city are such that it is advisable to employ combined sewers in all parts of the city down to the lowest contour line which will permit storm-water overflows to be used. This is the rule adopted by E. J. Fort for the new sewerage works of Brooklyn. Below this contour line, the storm-water sewers are run at a higher level than the house sewers, so as to have a free outlet to tide water, and the house sewage of the low districts is pumped to points of disposal.

In some cities the revision of old sewerage systems has been coupled with the protection of low-lying districts against flooding, as in Washington. In the original plan for the improvements, two levees with a total length of 4000 ft. were proposed for the protection of about 900 acres of water-front property, but later a large amount of filling of park and city property and raising of street grades was substituted for the original project. The city, which uses the combined sewerage system, now has intercepting sewers around it, and a few through it in order to take advantage of topographical conditions which enable the sewage of the higher parts of the city to be kept out of the low-lying parts. All the dry-weather sewage is delivered to a pumping station which discharges it through an outfall sewer 18,000 ft. long into the Potomac River about 800 ft. from shore. A considerable quantity of storm water from low-lying parts of the city is also pumped at this station, but only into the Anacostia River on the bank of which the plant is located.

After the most favorable location of the main lines of sewers has been determined, the desirability of minor changes of position in order to avoid needless interference with travel through busy streets should receive attention. The construction of a sewer in a narrow or crowded street costs the community a considerable sum in indirect damages and directly affects those having places of business on the street.

CLASSIFICATION OF SEWERS

Until quite recently there was considerable confusion in the terms used to designate different classes of sewers. A classification is necessary because it affords the only convenient means of discussing collectively the features of sewers for the same purpose in different parts of a system or in different cities, but the different classes necessarily run into each other somewhat so that no clear line of distinction between some of them is practicable.

House Drains, house sewers or house connections are the small pipe sewers leading from buildings to the public sewers. Strictly speaking, the house drain is the nearly horizontal piping in a cellar into which the soil and waste pipes discharge, but custom has extended the use of the term to the house sewer. In some cities they are put in and the connections with the public sewers are made by plumbers, but in other places the part of the work under the street as far as the property line, or even the whole drain from the sewer to the house, is laid by the city. City construction is advocated by many engineers on the ground that it is necessary in order to prevent injury to the sewers where the connections with them are made and to insure good workmanship on the drain in order to avoid digging up the streets to remove obstructions caused by poor construction. On the other hand where the municipal

regulations governing house drains are properly drawn and rigidly enforced by competent inspectors, there has been little complaint of the work of private contractors.

In most large cities so much trouble has been caused by the breakage of vitrified clay pipes in or near the place where they pass through walls that a rule has been issued requiring cast-iron pipe to be employed for the drain for a distance of several feet outside the walls. Even if cast-iron pipes are used, care must be taken to have them firmly supported so that they will not be cracked by settling. Where there is danger of a settlement of the foundations of the building, local conditions must determine the best construction.

The minimum size of house drains is 4 in., for smaller sizes are liable to become clogged frequently, but 5 in. or 6 in. sizes are considered better practice by many engineers, the latter being commonly adopted in the larger cities. The minimum fall for a drain is usually fixed by a city regulation, and less than 1/4 in. per foot is rarely permitted. Where the house drain must carry rain-water as well as house wastes, city regulations sometimes fix the size of the pipe by the size of the lot and an assumed rate of rainfall. In New York, for instance, the basis of calculation is a rainfall of 6 in. per hour with the drain running nearly full at a minimum velocity of 4 ft. per second. These figures lead to large drains in the case of buildings covering considerable area, and in such cases two or more drains are often run to the street sewer. The capacities of pipes are discussed in Chapter II.

Owing to the annoyance which may be caused by a stoppage of a house drain, just as much care should be paid to its location and construction as is given to a street sewer. It should run on a uniform grade and straight alignment, if possible, and where a bend must be made it is generally considered desirable to use curved pipe if the deflection is more than 6 in. in 2 ft. Some engineers recommend inspection holes at every angle in a house drain; these are shafts of small vitrified pipe rising from a tee in the drain, and they are objectionable because their weight often breaks the pipe below and their top is easily damaged by lawn-mowers and children. In any case there should be a clean-out hole on the drain just inside the house, where a cleaning-rod or heavy wire may be pushed into the drain to determine the location of, and if possible push along, any stoppage.

The house drain enters the sewer at a branch, if the sewer is pipe, or a slant if it is masonry. Where the sewer is in a deep trench, a vertical pipe called a chimney, encased in about 6 in. of concrete, is sometimes run up from every branch or slant by the contractor. It ends at a uniform depth below the surface, such as 13 ft. in the Borough of the Bronx, and the house drain is connected to its top. In any case the angle of the entrance should not be more than 45 deg., for the

splashing of the hot liquid house wastes containing grease on the cool walls of the sewer is liable to cause a heavy, tough coating on the latter, which reduces the discharging capacity, and this splashing will be less if the sewage enters at an easy angle than at 90 degrees. For the same reason, it is well to give only a moderate vertical angle to the inlet into the sewer, and to place the slants in brick sewers in such a position that they do not allow the house sewage to trickle over much of the wall before mingling with the dry-weather flow. A one-eighth bend may be used next the branch or slant in order to give the line a rectangular position with respect to the sewer.

In every case, care should be taken that the house drain is so constructed that there is no danger of sewage backing through it into the cellar.

Lateral Sewers.—The smallest sewers in streets are termed the laterals, and the extremities of the laterals are termed dead ends. Experience has shown, as explained in the Introduction, that preferably they should not be less than 8 in. in diameter on a separate sanitary system, for a smaller cross-section is liable to become clogged, although in small communities 6-in. pipes are sometimes used with success. There is a marked tendency to consider 12 in. as the smallest diameter for a combined sewer or storm-water drain. Theoretically anything liable to cause clogging should lodge in the house drains, but theory is not so good a guide as practice in this connection.

Manholes affording access to sewers are described in Chapter XIV. No sewer which is so small that a man cannot enter it should have any curve or change in grade between manholes, as otherwise cleaning it may be very difficult. Large sewers may be given such curves and changes in grade as conditions demand, but with small sewers the changes should be made by channels in the bottoms of the manholes, the loss of head due to the turning being compensated by an increased fall in the manhole. This increase is arbitrarily assumed by the designer, half an inch fall in the whole length of the channel in the manhole bottom being an amount often selected.

The depth of the laterals below the street surface should generally be as little as possible and still give adequate drainage to the houses. This depth varies greatly, for in a city like New Orleans few houses have cellars and therefore shallow depths are sufficient, whereas in Boston and New York deep cellars prevail and consequently the sewers must be still lower. Where a sewer is laid in a street running along a steep hillside, it sometimes has to be given unusual depth to receive the sewage from the lower side. In suburban districts with houses set well back from the streets, it is not uncommon for a house to remain connected with a cesspool or subsurface irrigation system, because of the impracticability of making a workable connection with the sewer which will serve

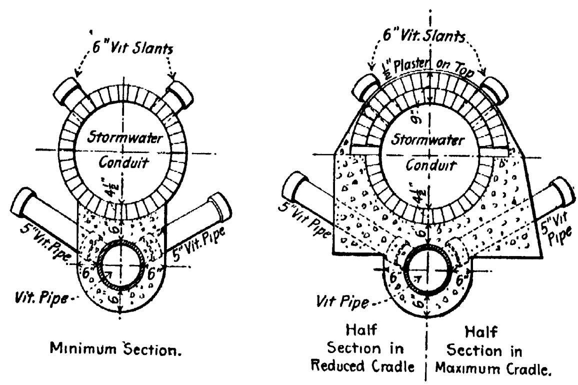
adequately neighboring houses. One long sewer in a street is often more expensive than two short ones having the same total length but discharging in opposite directions. The long sewer is likely to have a flatter grade and to require a deeper trench at its lower end. Where the territory is flat and the ground-water table close to the surface, it may be necessary in order to give the laterals sufficient fall, to construct them below the water table; in such a case, risers are sometimes run up so that the house drains can be connected to them above the ground-water level.

The proper capacities and the minimum grades of sewers are discussed in detail in subsequent chapters. It is only necessary to state here that the true grade of a combined sewer or storm-water drain is its hydraulic gradient. There has been an unfortunate tendency to use for computations the grade of the invert or the crown of the arch as the grade of both combined and separate sewers, a practice which has led to serious trouble in some cities.

There is always some uncertainty regarding the amount of ground water which will leak into a sewer and a large group of small buildings or a schoolhouse will concentrate at one locality a quantity of sewage which cannot be foreseen. The velocity when the sewer is half full is as great as when it is full, consequently, there is no greater probability of sedimentation of solid matter. The difference in cost between small sewers of different diameters is not great and consequently in order to obtain satisfactory velocities with the volumes of sewage which may reasonably be expected, and still have capacity for an occasional unexpected condition, lateral pipe sewers are sometimes figured as running half full when carrying the maximum quantity of sewage which it is assumed will reach them. Some engineers have continued this policy until pipes as large as 18 in. have been reached and in computing this size they have provided for a depth of only seven-tenths of the diameter, using larger sizes when the quantity to be carried exceeds the capacity at this depth and in each case using seven-tenths of the diameter as the position of the hydraulic gradient. It appears to the authors to be more logical to make allowances for such unusual increments in flow when determining the maximum quantity to be provided for, bearing in mind that the size of pipe must be such as to provide self-cleaning velocities under usual conditions of flow, and to figure the sewers as running full.

Where the separate system is used and a large storm-water drain runs through a street, it may be very difficult to connect the houses with a single lateral, and it is sometimes advisable in such cases to run a lateral on each side of the street as in Washington. This places an additional conduit in the street, but it eliminates a large amount of troublesome work with small pipes crossing the street, which will interfere with the laying of future conduits. If the street is wide and the lots have a small frontage, the double laterals may even be cheaper.

While the position of the laterals in the street is influenced by local conditions they are usually placed in the center thus equalizing the length and cost of house drains which are built wholly or in part by the abutting property owners. This location favors a minimum depth of sewer to provide proper fall for the house connections. As sewers are usually laid quite deep in comparison with water and gas mains, they should be kept at least 6 ft. from the latter, if possible, so as to avoid the danger of injuring them during construction. Where the line is on one side of the street and property owners pay for the actual length of their house connections, those on one side have a financial advantage over the others, which can be remedied, where the drains are laid by the city, by assuming that in every case the drain runs to the center of the street.



Side manholes affording access to the sanitary sewer from the side instead of the top are used in this form of construction.

FIG. 4.—Standard arrangement of separate sewers, Philadelphia.

In Philadelphia, standard general sections for installations on the separate system have been adopted by George S. Webster, Chief Eng. of the Bureau of Surveys. The relative position of all conduits under 3 ft. diameter is shown in Fig. 4; the general arrangement for larger conduits is much the same. Slants and pipes for house connections are put in every 15 ft. The minimum thickness of concrete between the conduit and pipe is 6 in., except in rock excavation. With these sections the filling over the top of the conduit is at least 3 ft. deep. Many engineers prefer to have the sewers at one side of the drains, in order that they may be reached readily; this requires a wider trench than where the two-story arrangement of Fig. 4 is employed.

Branch Sewers.—The lateral sewers frequently discharge into long branches, which in turn discharge into the trunk sewers. Experience

has indicated that these long branches, which lie on the boundary between pipe and masonry construction, are quite troublesome to arrange, and that defects in their plans are as likely to arise as in the design of the other classes. The reasons for this are several. In order to economize in the cost of construction of both sewers and house drains, the depths of the sewers below the surface should be as small as possible, but in order to carry off the sewage from the laterals the branches must necessarily be deeper than local house drainage alone demands. The grade must be steep enough to give an adequate scouring velocity and flat enough to keep the points where the branches enter the trunk sewers high enough to allow the latter, in the case of combined sewers, to discharge their dry-weather flow into intercepting sewers. Furthermore branch sewers serve relatively small districts, and if storm-water is carried by them, a material increase in the extent of impervious territory may make such a change in the maximum amount of run-off reaching them in short periods of time that they will become surcharged before the capacity of the large trunks is reached. On the other hand, branch sewers of large capacity but carrying small quantities of sewage are likely to collect sludge on the inverts, owing to the low velocities. Consequently the engineer has to select a size and grade which reduces the total of disadvantages to a minimum. In such cases the egg-shaped sewer is sometimes employed to advantage, owing to the small channel at the bottom of the section, which usually has a radius of about one-fourth the maximum width. The total height of the sewer is generally about one and a half times the maximum width.

In many cases it is impracticable to connect the laterals to the lower portion of a branch without using very deep trenches for the lower parts of the laterals and their house drains, or else keeping the lateral at a higher elevation and allowing them to discharge into the branch sewer through a drop manhole, a special structure described in Chapter XIV. The choice between the deep lateral or the drop manhole depends primarily on their relative cost, and in determining costs the expense of deep house connections as well as laterals should be considered.

It was pointed out by Dr. Hering in 1881 that an axiom of sewerage design was that a sewer of X times the capacity of another does not cost X times as much money, and it is therefore desirable to lead as many laterals together into branches as possible. This also gives the laterals better grades, as a rule.

Another thing to be considered with low-lying sewers in districts where high buildings are carried on wood piles was brought out as follows in a report on the sewerage of Hoboken, made in 1912 by James H. Fuertes:

"Many of the large and fine buildings in Hoboken rest upon wooden piles, and these will remain safe and stable so long as the piles are kept submerged below the ground-water level. If the ground-water level were to be lowered

below the present prevailing height, then trouble would be sure to be felt in a comparatively short time, by the rotting of the piles and grillages, the crushing of the timber and the settlement of the buildings. If all the sewers and their connections were perfectly tight and would remain so, there would be little likelihood of danger from this cause in securing good deep-cellar drainage. I am quite certain, however, that sewers cannot be maintained in such a condition in Hoboken."

This recommendation is confirmed by observations in New York, where the construction of subways and sewers has lowered the ground-water level in places and comparatively new foundation piling has rotted away.

Trunk Sewers.—The trunk sewers are the main stems of the sewerage network; in small cities there may be only one, but in large cities there may be several, sometimes uniting where the general arrangement of the system is that of a fan and sometimes discharging independently into rivers, lakes or ponds, like the trunk combined sewers of New York and most storm-water drains everywhere.

There is, of course, a great difference in the design of the trunk sewers of separate and combined systems. Where storm water enters into consideration, it usually exceeds the amount of house sewage so greatly that the capacity of the sections is fixed by it. The only influence of the house sewage on the design is to govern to some extent the shape of the invert, in order that the channel for the dry-weather flow may be such that the velocity during rainless periods will be maintained within desirable limits. The flow in sewers is discussed in the next chapter.

The size of trunk sewers receiving house sewage only may be selected on somewhat narrower lines than the size of the laterals and smaller branches, because it is hardly probable that all these small sewers will receive more sewage than the expected future maximum. Nevertheless in many cases the maximum assumed quantities are not more than about seven-tenths of the greatest capacity of the sections provided.

Where trunk sewers lie deep and the branches discharging into them would naturally be much higher, well-holes are sometimes used to connect the two. These devices are described in Chapter XIV on special structures, which also gives a description of flight sewers, occasionally required where a heavy drop in the grade of a trunk sewer is necessary.

A feature of design which should be mentioned in this place was stated as follows in Dr. Hering's report to the National Board of Health in 1881:

"The junction angle of converging sewers should be arranged so that the direction of flow of the two streams before joining is as nearly as practicable the same. Neither will then lose much velocity in endeavoring to overcome the change in direction. The less the sizes of the respective streams differ from each other the more essential is this consideration. An important feature of junctions is the relative height of the joining streams, for unless

this point is considered backwater and deposits may occur in one of them. Theoretically the joining sewers should be so shaped as to constantly deliver the sewage of each at the same level. To comply with this demand on all occasions is impossible, and it will suffice to consider the ordinary flow which occurs during nine-tenths of the time. The surface of the latter in the branches should be either the same for all, or increase in height as the bulk of the sewage becomes less. In other words, the smaller sewers should join the larger ones so that their ordinary flows meet at the same level, or so that the smaller sewer discharges at a higher level. When two sewers discharge into a manhole opposite to each other, at points above its bottom, they should be placed at different heights, or else receive a slight lateral turn, so that the full discharges do not directly meet each other."

Trunk sewers on the combined system are such expensive works that every opportunity should be sought for reducing their cost legitimately. Sometimes this can be done by providing several points where surplus storm water can escape through short channels or conduits to neighboring bodies of water, and at London provision has even been made to pump some of this storm water into the Thames rather than give the long trunk sewers the size needed to handle it. These pumping stations are operated by gas engines, and are run only when the storm water must be handled. Sometimes the first cost of combined trunk sewers can profitably be reduced, where the cost of construction is not heavy, by employing a rather small cross-section and constructing another trunk sewer later when it is needed. Where construction is expensive on account of poor ground or the presence of large amounts of water, or imposes a serious burden on the business of the streets in which it is carried on, it is usually advisable to design the trunk sewers to serve the community for the entire period which the interest rates on the cost of the sewers make most economical. Often the most satisfactory method of keeping down the cost of combined trunk sewers is to run them to the nearest bodies of water and draw off the dry weather sewage into intercepting sewers near their lower ends. The extreme lower ends of the trunk sewers thus discharge storm water during rains while at other times the house sewage passes into the intercepting sewers. The methods of delivering the sewage into the intercepting sewers are explained in Chapter XVI, on the design of special structures.

Intercepting sewers, or collectors, are of two distinct types. The first receives part or all of the sewage of the system above a given contour, and is employed either to permit a reduction in the size of the intercepters at lower levels or to discharge by gravity the sewage from districts high enough to make pumping unnecessary. In the latter case low-level intercepters, which are really trunk sewers although custom does not give them that name, are employed to convey the sewage from the low lands to the pumping stations. The second type of intercepter crosses below the trunk sewers of combined systems and receives the dry-weather sewage carried by them. By restricting its duty mainly to the house sewage,

it can be kept of relatively small size and the sewage can thus be conducted by it in the most economical manner to the place of disposal. By a suitable allowance in the design of the special structures for intercepting the house sewage, the offensive first wash of the storm may also be diverted.

Interceptors are given capacities determined by the methods explained in Chapters V and VIII. They generally carry from 300 to 400 gal. per person from a population estimated to exist from 30 to 40 years after the date of the designs.

Relief sewers are built to take part of the sewage from a district where the trunk or intercepting sewers are already overcharged or are in danger of becoming so. They may be used to take excess storm water where it threatens to surcharge old sewers, as happens when the area of impervious land increases greatly or additional territory is drained into these old main lines, or they may be made to serve constantly a given district and be connected with the branches and laterals in it, so as to restrict the service of the older trunk sewers to a more distant district. Experience in large cities, notably in London, shows that more than one relief sewer may eventually become necessary for a given district.[1]

The construction of relief sewers is not necessarily an indication of any error in the original plans of a sewerage system. As already stated, it may be wise under some local conditions to use rather small trunk sewers at first, particularly if there is considerable doubt as to the direction in which the city's population will extend. If funds permit and the difficulties of construction are great, it is best as a rule to provide ample capacity.

Outfall sewers are the large lines leading to the places of disposal. The end of an outfall sewer running into water is termed its outlet; "outfall" is sometimes used for "outlet." The discharge of a sewer which is partly or wholly submerged is discussed in the next chapter.

Inverted siphons are sewers which run under pressure due to their dropping below the hydraulic grade line and then rising again. The name is a poor one, but not so bad as "siphons," which is occasionally employed, although that term means something entirely different. It would be much better to speak of all such sewers as pressure sewers.

[1] The admission of the runoff from roofs into separate sewers may prove the cause of an early necessity for greater sewerage facilities in a district where this practice is permitted. For example, in some of the sections of Cincinnati, which were provided with separate sewers before they were annexed, the runoff from roofs was permitted to be discharged into the sewers. After those sections of the city became built up to a greater extent, this runoff overtaxed the sewerage capacity, and storm sewers were added as street improvements were made. In cases like this, relief sewers of districts provided with sanitary sewerage systems become absolutely necessary and it not infrequently happens that the existing sanitary sewers are retained for that purpose exclusively and combined sewers of large size are built to act as trunk sewers receiving the discharge from the separate sewerage system and also from storm-water drains.

They are most frequently employed to cross under rivers, but occasionally are needed on outfalls to avoid the long lines which would be required to keep the sewers on the hydraulic gradient, or to make pumping unnecessary.

In their design it is necessary to allow for internal pressure, and until recently cast iron or steel pipe has generally been employed for them. With the development of reinforced concrete, however, a new material has become available for pressure sewers built in the trench, which have been constructed of noteworthy dimensions in Paris, and still more recently reinforced-concrete pipe of large size have been made and tested under pressures up to 90 lb. per square inch, by the Lock Joint Pipe Co. The difficulty previously encountered with such pipe under pressure has been in the joints, but in the tests mentioned (see *Engineering News*, Dec. 4, 1913) a special joint was employed in some cases and this proved tight. This type of pipe is described in Chapter X, and has been adopted for pressure service in the Baltimore water works.

The various details at the ends of inverted siphons are described in Chapter XV. In any case where such siphons are employed, care should be taken to provide blow-offs at the lowest points, if possible, and to prevent, so far as practicable, coarse materials from entering them. In connection with such a pressure sewer at Fitchburg, Mass., for example, a large grit chamber with screens has been provided, and a blow-off branch has been built to the Nashua River.

Force mains are pressure sewers through which sewage is pumped. Where small pumping stations are used to avoid placing sewers in deep trenches, it is often desirable to concentrate the lift at the stations, the sewage flowing to them by gravity and, after being lifted, flowing away by gravity, thus avoiding the use of a long force main.

Flushing sewers are occasionally used in sewerage work to flush out water-courses receiving sewage or to convey water for flushing to the head of the lines to be kept clean. They are not sewers, strictly speaking, but water conduits. Milwaukee, Chicago and Brooklyn possess flushing works of the first class. A good example of the second class was proposed by James H. Fuertes in 1912 for use in connection with new sewers at Hoboken, N. J. This plan called for large shallow reinforced-concrete tanks at the heads of the flat trunk sewers needing flushing. The tanks are to be supplied with harbor water through pipe flushing sewers built into the concrete foundations of the main sewers, a flap valve being placed on the end of the supply pipe in each tank. In this way the tanks will be filled on rising tides and the flap valves will prevent the escape of the water as the tide falls. At the proper time on the falling tide, a sluice gate will be opened automatically and quickly to let the water run out of the tank into the sewer, the operation of the gate being controlled by a float.

GENERAL DETAILS OF SEWERAGE SYSTEMS[1]

Grades.—Although the grade of the invert is usually meant when the grade of a sewer is mentioned, in determining the cross-sections of combined and storm-water sewers the surface of the flowing sewage or the hydraulic gradient should be the controlling grade. In the case of separate sewers for house sewage alone, this distinction is rarely important and consequently is generally disregarded, but with combined sewers, where the surface of the water in the sewer during heavy rains may have a smaller slope than the invert, the surface gradient must be the controlling inclination or unpleasant conditions may arise like those which existed in Brooklyn, as mentioned in the Introduction.

The invert grade is the most important factor controlling the flow in sewers carrying only house sewage, and in combined sewers while only the dry-weather sewage is flowing. As explained in detail in the next chapter, the slope, s, is equal to v^2/c^2r, where v is the velocity, c is an empirical coefficient and r is the hydraulic mean radius or the area of the cross-section of the flowing stream divided by the length of the portion of the perimeter of the section which the water touches. As it is apparent that at very low depths, there must be some uncertainty regarding the accuracy of the formula's results, some assumption of a minimum depth of the stream to which it is applicable must be made; this is taken at about 0.8 in. in Germany. Less than this results in the stranding of suspended matter on the invert until it is flushed out by a larger flow than usual. In the case of the smallest laterals, it is inevitable for them to be dry near their dead ends at times, and a mere trickle of sewage generally flows through them, so that the stranding of suspended matter in them is common and they are often kept clean by flushing, either by hand or by automatic apparatus described in Chapter XV. As a result of experience and observation, American sewerage specialists have reached a fairly uniform practice in respect to minimum grades for these small sewers, which is explained in detail in Chapter III. A rule for the minimum grade much used in England is to make it equal to $1/(5d + 50)$, where d is the diameter in inches. In Germany circular house sewers with a diameter of 4 to 5 in. are given slopes of 1:15 to 1:30, if possible; house sewers of 6 in. diameter, slopes of 1:20 to 1:50; lateral sewers up to 12 in. diameter, slopes of 1:30 to 1:150, and from 12 to 24 in. diameter, slopes of 1:50 to 1:200. With egg-shaped sections, the minimum slopes are somewhat reduced; the preferred range of grade of branch sewers of such a section is from 1:100 to 1:300. In large trunk

[1] In this subchapter, the authors have adopted many of the methods of presenting the subject which are found in Frühling's "Die Entwässerung der Städte," fourth edition, 1910, a treatise embodying the results of the investigations and studies of one of the foremost German sewerage specialists.

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this arrangement does not help the unfavorable condition in the main sewer.

A special condition arises in combined sewers where there is a relief outlet. When a large amount of storm water is flowing and the outlet is in operation, Fig. 8, there is an increase in the hydraulic gradient for

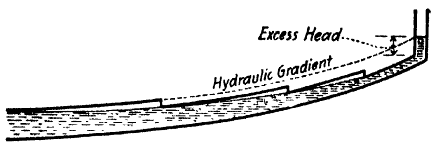


Fig. 5.

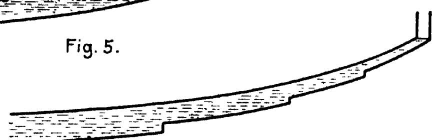


Fig. 6.

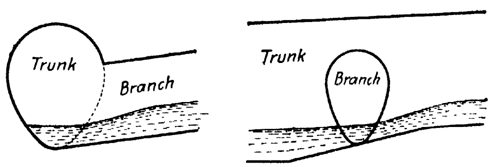


Fig. 7.

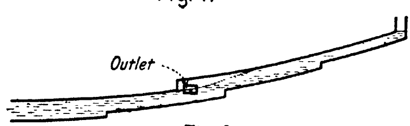


Fig. 8.

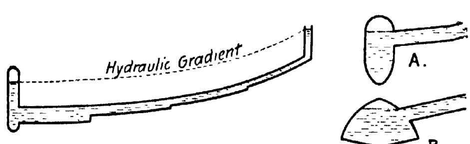


Fig. 9.

Figs. 5 to 9.

some distance above the outlet. Moreover, in the part of the sewer affected by this change in the hydraulic gradient, the entering branches are also similarly affected and there is a corresponding general increase in velocity. This fact is rarely taken into consideration, nor an unfavorable consequence of it if the sewers are not kept clean, viz., the picking up and sweeping along of sludge previously deposited above the outlet.

A frequent cause of congestion in a branch sewer, and its attendant surcharge, is indicated in Fig. 9, where an increase in the elevation of the branch and its tributary lines is impracticable on account of the local conditions. The surcharge of the branch can be avoided in this case by lowering the trunk sewer, as in A, or it can be at least greatly reduced by employing a wider section, as in B, which will lower the hydraulic gradient. It is also possible to give the branch sewer a larger section, high and narrow, and thus reduce its hydraulic gradient, but this is expensive and it may prove best to build the lower stretch of the branch very carefully with the object of making it carry internal pressures at times.

In laying out a combined sewerage system it is evident from what has been said that it is usually first necessary to determine the minimum permissible elevation of the sill of the lowest relief outlet. This will enable the elevation of the trunk sewer at that point to be established, and from that elevation the grades of the upper portion of the system can be worked out. The best location of the various lines can only be determined by a number of trials, in many cases, and the failure to give proper study to grades and hydraulic gradients has been the cause of much of the unsatisfactory service of sewerage systems. The work is not unlike, in some respects, the location studies of railway lines, which have also frequently been hurried along, to the great disadvantage of the subsequent operation of the roads.

Relief Outlets.—Relief outlets for the escape of storm water from large sewers into nearby rivers or lakes are an essential feature of any system of combined sewers, for otherwise the trunk sewers would require enormous dimensions. In rare cases, as in New Orleans, it is necessary to collect and pump all the storm water and under such conditions a separate system with independent drains for removing the rainfall is the only solution of the sewerage problem. The purification of all the rain-water of a city has never been considered necessary, and the problem is to determine what dilution of the house sewage with rain-water is desirable before the mixture may be discharged through the relief outlets.

There will be some sewage escape into the river or lake whenever there is a discharge through one of these storm overflows. If the sewers are not kept clean, the amount of organic matter which is discharged in this manner will be higher than otherwise, because the scouring action of the storm water in the sewers will sweep it from the inverts where it has settled during dry weather. But as many rainfalls will not yield enough water to bring the storm overflow into service, although they will increase the flow in the sewers enough to take up some of the deposits on the inverts, it is apparent that with well-designed and built sewers, the uncertainty as to the degree of dilution of the house sewage during heavy storms will be unimportant in most cases. The relief outlets do not usually discharge often enough in a well-designed system to make the amount of organic

matter escaping through them into the river of significance as respects the condition of the latter.

There has been a great difference in the ratio of the storm water to house sewage adopted as the basis for the design of the relief outlets. It is naturally larger when the outlet discharges into a small sluggish stream than where there is a larger body of water to receive the excess quantity. If the outlets are along a river and it is more desirable to keep its upper course uncontaminated than its lower course, the storm overflows along the latter should be much larger than the others, even though this makes it necessary to employ larger trunk sewers than would otherwise be necessary between the first and last points of relief.

The value of the ratio has ranged from about 2 to 8. The phenomena that take place in a sewer during the period when the overflow is in service have not been investigated so fully as is desirable. As already explained, there is an increased velocity of flow when the outlet begins to discharge, and this results in a somewhat larger volume of sewage continuing in the trunk sewer than the usual computations make allowance for. Furthermore the discharge of a weir parallel to the thread of the current may not be so great as when the weir is at right angles to the current.

Numerous relief outlets have the dual advantage of keeping down the size of the sewers and discharging the excess storm water at several places rather than concentrating it at one. The cost of the outlet conduits from the overflows to the points of discharge, as compared with the cost of sewers of different sections, will afford a useful guide to the best number. Old sewers and the channels of brooks can sometimes be utilized to advantage as the outlet conduits.

The design of these overflows is described in Chapter XVI.

The discharge over the sill of a relief outlet depends on the elevation and length of the sill, the shape of the outlet and the dimensions of the main sewer above and below the outlet. Inasmuch as there is no direct experimental knowledge of the discharge of weirs parallel to the direction of the current and other conditions of the case are unlike those favorable to fairly true results from the use of the standard formulas for weir discharge[1] (which are discussed in Chapter IV), Frühling advises for use in computing the discharge: $Q = 4bh^{1.5}$, where Q is the quantity in cubic feet per second, b is the length of the sill in feet and h is the depth in feet of water over it. Another method of estimating the discharge is given in the chapter on the design of relief overflows. It is more elaborate but whether it gives results which approach the truth more closely is a matter of guesswork in the absence of reliable experimental information.

[1] "He had been trying to get information with regard to some of the long overflows on the sewers in London—some of them 30 ft. in length—but he found it very difficult to do so. In some cases nearly all the water flowed past the overflow-weir until the water was considerably higher than the level of the weir, so that the full effect of the overflow was not obtained."—Maurice Fitzmaurice, Proc. Inst. C. E., vol. clxiv, 61.

If the relief conduit is so designed that its lower end is completely closed by high water in the river or lake into which it discharges, the hydraulic gradient of the conduit should be investigated to make sure that backing-up of the water in the conduit will not interfere with the free action of the weir.

Below the relief outlet the trunk sewer carries a smaller quantity of sewage than above it, and, with the same grade, it may be given a smaller cross-section. With an increase in elevation of the sill of the overflow there is an increase in the quantity of water which remains in the trunk sewer. A long sill is better than a short one for regulating the quantity of water which escapes and, consequently, the quantity which remains in the sewer.

If, for any reason, the sill of the storm overflow must be placed so low that the floods in the river rise above it, but not to the crown of the trunk sewer, the discharge of the overflow will then be checked. There are no published observations of what the discharge will be under such conditions, but from Herschel's discussion of the flow over submerged weirs (Trans. Am. Soc. C. E., XIV, 194) and adopting only two-thirds of his quantities, the volume of sewage escaping from an overflow under such conditions will probably not fall below an amount given by the expression $Q = nbH^{1.5}$, where Q is the discharge in cubic feet per second, b is the length of the sill, H is the depth of the sill below the water surface in the sewer and n is a coefficient taken from the following list and depending upon the ratio of h, the depth of water in the relief channel from the surface to the sill, to H.

h/H. . . .	0.1	0.2	0.3	0.4	0.5	0.6	0.7
n.	2.2	2.2	2.1	2.0	1.6	1.3	1.1

If the college laboratories having facilities for making such experiments will determine approximately coefficients which may be safely used for both free and submerged weirs like those used for relief outlets, the information will prove of much practical value. Until such investigations are made, the designer must fix the lengths of the sills by the methods indicated, or others equally approximate.

Preliminary Studies.—In making the preliminary studies of a system of sewers, it is sometimes customary to use merely tables of the discharge of sewers laid on a grade of 1 per cent. Tables 3 and 4 are examples, based on a value of $n = 0.013$ in the Kutter formula, explained in the next chapter. Some engineers prefer to use such tables and a slide-rule to reading quantities from diagrams like those given in the next chapter, and to illustrate their use as well as to introduce at this point some of the more general problems arising in sewerage work, a few examples of preliminary studies (adapted from Frühling's "Entwässerung") are

given here. The basic fact to be kept in mind is that velocities and discharges vary about as the square roots of the grades.

1. A sewer 1476 ft. long with a fall of 6.56 ft. must discharge 4.097 cu. ft. per second; what should be its diameter and velocity?

The average slope is 6.56/1476 = 1/225. The tables are prepared for slopes of 1/100; velocities and discharges for other slopes vary as the square roots of the slopes. The discharge on a slope of 1/100 corresponding to 4.097 sec.-ft. on 1/225 is $4.097\sqrt{(225/100)}$, which is readily found by a slide-rule to be 6.15 sec.-ft. If it is desired to have the sewer run full when discharging, Table 3 indicates that a 15-in. circular section will be correct, and the velocity will be about 3-1/2 ft. per second. Egg-shaped sections are too expensive for discharges as small as this. The velocity with smaller discharges may be found by dividing the tabular velocities for the different depths of sewage by $\sqrt{(225/100)}$. It is evident that the velocity sinks to 2-1/2 ft. when the sewage has a depth of less than about 5 in.

2. The 15-in. sewer of Ex. 1 discharges 0.053 cu. ft. per second during dry weather into an egg-shaped sewer 69 in. high on a grade of 1:1200, carrying 0.88 sec.-ft. of house sewage; what is the best way to prevent backing-up of sewage at the junction?

The discharge of the main sewer with the same depth of flow and a slope of 1:100 will be $0.88\sqrt{(1200/100)}$ or 3.06 sec.-ft., which Table 4 shows will fill less than 0.1 of the depth of the section or, say, 6 in. In the same way the depth of flow in the 15-in. sewer with 0.053 sec.-ft. may be found to be less than 0.1 of its diameter, or, say, 1-1/2 in. Hence, as a first approximation, it may be assumed that the invert of the lateral must be 6-1-1/4 = 4-3/4 in. above the invert of the main branch to cause the surface of the sewage in the two sewers to be at the same elevation. This results in a loss in the invert grade in the lateral, which is not likely to be of importance except where the available fall or slope is restricted. The discharge of 0.053 into 0.88 sec.-ft. will cause only a trifling increase in depth and loss of velocity in the main sewer. After the general layout has been worked up approximately, the elevation of the branch sewer at the junction may be readjusted by the more accurate methods explained in the next chapter.

3. The main sewer of Exs. 1 and 2 is assumed to be two-thirds filled; what will be the effect of this condition on the lateral?

Two-thirds of 69 in. is 46 in.; (46-4-3/4) in. must therefore be taken as the total drop of the invert of the sewer in obtaining the grade for computing the discharge during such conditions. In other words, instead of a grade of 1/225, which is proper for calculating the discharge of dry-weather sewage, one of (6.56-3.44)/1476 must be used.

4. In the main sewer, 656 ft. below the junction of Ex. 2, there is a relief outlet with the water on its sill 33 in. above the invert during a storm; what effect will it have at a point 2132 ft. above?

This may be approximately solved by dividing the 2132-ft. length into several parts and assuming the hydraulic gradient to be constant in each stretch. The starting point is the elevation of the water on the sill, 33. The sewer flowing full will carry $211\sqrt{(100/1200)}$ = 61 sec.-ft. As the outlet is approached the hydraulic gradient increases, as mentioned earlier;

at the upper end of the outlet this quantity of sewage is carried in the bottom 33 in. of the section, or at a depth of about 48 per cent. of the height. The discharge of such egg-shaped sewers at different depths, as will be explained in Chapter III, and on p. 393, varies about as follows:

Depth..........	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
Discharge........	0.02	0.07	0.15	0.27	0.42	0.58	0.75	0.92	1.05	1.00
Velocity.........	0.41	0.61	0.75	0.85	0.95	1.05	1.08	1.11	1.11	1.00

At a depth of 48 per cent., the discharge will therefore be about 39 per cent. of the discharge of a full section at that velocity. Therefore 61 sec.-ft. divided by 0.39, or 156 sec.-ft. would be discharged at this velocity were the sewer full. The hydraulic grade is, therefore, $(156^2/211.1^2)(1/100) = 1/184$, at the outlet. The length of the sections into which the sewer is subdivided to ascertain the hydraulic gradient, may be taken of any length, as 164 ft., for instance. Thus, by the methods just explained, the heights of the points on the hydraulic gradient will be found as follows:

$$\text{Outlet} \qquad = 33.0$$

$$\text{Point 1, } 33.0 + (12 \times 164)\left\{\frac{1}{184} - \frac{1}{1200}\right\} = 42.1$$

$$\text{Point 2, } 42.1 + (12 \times 164)\left\{\frac{1}{420} - \frac{1}{1200}\right\} = 45.1$$

$$\text{Point 3, } 45.1 + (12 \times 164)\left\{\frac{1}{523} - \frac{1}{1200}\right\} = 47.2$$

$$\text{Point 4, } 47.2 + (12 \times 164)\left\{\frac{1}{625} - \frac{1}{1200}\right\} = 48.7$$

$$\text{Point 5, } 48.7 + (12 \times 164)\left\{\frac{1}{692} - \frac{1}{1200}\right\} = 50.1$$

It is evident from these figures that the effect of the outlet extends far above the 2132-ft. stretch and also affects the branches. The curve is so flat that it is unnecessary here to calculate more points on it; for approximate purpose it will answer to assume $63/(12 \times 164)$ as the average grade for the remainder of the stretch.

5. A flow of 44.14 sec.-ft. must be carried by a sewer with an invert grade of 1:900. The height of the sewer connecting with it above must not exceed 48 in., owing to the low elevation of the surface, and there must be no internal pressure. What egg-shaped section should be selected for the 1:900 grade?

Since discharges are proportional to the square roots of the slopes, a discharge of 44.15 sec.-ft. on a 1:900 grade is equivalent to one of $44.15\sqrt{(900/100)} = 132.45$ sec.-ft. on a 1:100 slope. Table 4 shows that a 60-in. section will carry this quantity, but the latter will require such a large proportion of the total capacity that there is danger of placing the next sewer above, under an internal pressure of $60 - 48 = 12$ in. To avoid this

it is better to employ a 69-in. section running two-thirds full, that is, with the sewage at an elevation of 46 in.

6. Owing to topographical conditions, a trunk sewer must have the profile shown in Fig. 10. What are the cross-sections and hydraulic gradients for the given invert grades and quantities?

The computation begins with the lowest stretch of sewer. The equivalent discharge on a 1:100 grade is $204.82\sqrt{(1000/100)} = 648$ sec.-ft., an amount so large that an aqueduct section of the semi-elliptic, semi-parabolic, segmental, horse-shoe or other type, described in Chapter XII, will be preferable to the egg-shaped, which would have a needlessly great depth, and consequently expense, to be able to carry such a quantity.

If the next stretch were to run full with the quantity stated on the profile, it would operate under the head due to the hydraulic gradient *ab*, which would probably be continued farther back up the line. In view of the abundant grade, the alternative arrangement at *a*, with a drop of some sort, such as a flight sewer or well-hole, is preferable. The invert and hydraulic

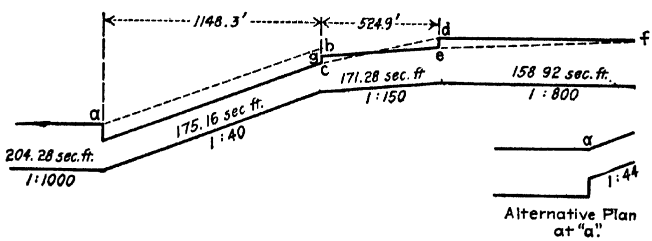


FIG. 10.

gradients are determined by trial, assuming for a first approximation a 60-in. section and that the lowest stretch is a semi-elliptical section 90 in. high. Then

$$\frac{1}{1148.3}\left(\frac{1148.3}{40} - \frac{90}{12} + \frac{60}{12}\right) = \frac{1}{44}$$

The volume of water will be $175.16\sqrt{(44/100)}$ or 116 sec.-ft. on a 1:100 grade, which corresponds to a section between 54 in. and 57 in. high. If the latter is chosen the hydraulic gradient will remain within the sewer even with the next section above running full.

Either *ge* or *cd* can be taken as the hydraulic gradient for the next stretch. In the latter case the sewer will require a somewhat smaller cross-section, but the upper part will be subject to an internal pressure of an amount depending on the height of the cross-section of this stretch, which it is therefore desirable to ascertain. An egg-shaped section, with a hydraulic gradient coinciding with the crown of the sewer *df*, will be assumed. The actual discharge and slope are equivalent to $158.92\sqrt{(800/100)} = 450$ cu. ft. per second on a 1:100 slope. This quantity is beyond the scope of Table 4, but with the help of Fig. 27 of Chapter II it will be found that a section about 94 in. high will suffice. If it is assumed tentatively that the hydraulic

TABLE 3.—VELOCITY IN FEET PER SECOND (V) AND DISCHARGE IN CUBIC FEET PER SECOND (Q) OF CIRCULAR SEWERS WITH DIFFERENT PROPORTIONS (H) OF THE DIAMETER (D, I

AND A GRADE OF 1:100

(Based on n = 0.013 in Kutter's Formula)

4-in.		6-in.		8-in.		10-in.		12-in.		15-in.		18-in.		20-in.		22-in.		24-in.		30-in.		33-in.		36-in.		42-in.		
V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	
0.65	0.0052	0.88	0.015	1.15	0.036	1.36	0.068	1.68	0.113	1.85	0.189	1.93	0.311	2.28	0.416	2.45	0.528	2.61	0.682	3.06	1.249	3.26	1.615	3.46	2.04	3.85	3.09	4
1.14	0.0142	1.41	0.045	1.79	0.099	2.11	0.185	2.61	0.306	2.87	0.567	3.28	0.934	3.55	1.247	3.81	1.574	4.05	2.045	4.75	3.74	5.08	4.84	5.39	6.13	6.00	9.27	6
1.30	0.030	1.82	0.094	2.30	0.201	2.71	0.391	3.35	0.647	3.69	1.196	4.22	1.97	4.56	2.63	4.90	3.41	5.21	4.34	6.11	7.81	6.53	11.08	6.93	12.96	7.71	19.6	8
1.59	0.053	2.22	0.163	2.80	0.369	3.32	0.686	4.10	1.135	4.51	2.10	5.16	3.43	5.58	4.62	[illegible].88	5.98	6.37	7.58	7.43	13.89	7.98	17.95	8.48	22.7	9.43	34.3	10
1.811	0.0790	2.521	0.2475	3.168	0.553	3.770	1.028	4.335	1.702	5.131	3.148	5.872	5.190	6.343	6.9[illegible]	6.797	8.97	7.237	11.365	8.481	20.81	9.067	26.92	9.63	34.04	10.71	51.50	11
1.95	0.106	2.72	0.332	3.44	0.741	4.07	1.378	5.03	2.28	5.54	4.23	6.32	7.96	6.85	9.[illegible]	7.32	12.02	7.82	15.22	9.16	27.9	9.80	36.1	10.40	45.6	11.56	69.0	12
2.06	0.133	2.87	0.411	3.64	0.966	4.30	1.728	5.31	2.86	5.82	5.29	6.68	8.62	7.23	11.[illegible]	7.75	15.07	8.25	19.10	9.57	35.0	10.34	45.2	10.98	57.2	12.21	96.4	13
2.10	0.155	2.92	0.485	3.70	1.083	4.37	2.03	5.41	3.33	5.95	6.17	6.79	10.18	7.35	[illegible]	7.86	17.58	8.40	22.4	9.84	40.7	10.52	52.8	11.17	66.7	12.42	100.9	13
2.08	0.170	2.90	0.530	3.67	1.188	4.34	2.21	4.99	3.65	5.90	6.75	6.74	11.12	7.29	[illegible]	7.80	19.26	8.33	24.4	9.78	44.7	10.43	57.8	11.07	73.1	12.32	110.6	13
1.811	0.1580	2.521	0.4950	3.168	1.106	3.770	2.057	4.335	3.405	5.131	6.296	5.872	10.38	6.343	[illegible].85	6.797	17.94	7.237	22.73	8.481	41.63	9.067	53.85	9.63	68.09	10.71	103.00	11

E 4.—VELOCITY IN FEET PER SECOND (V) AND DISCHARGE IN CUBIC FEET PER SECOND (Q) OF EGG-SHAPED SEWERS OF THE TYPE SHOWN IN FIG. 10, WITH DIFFERENT PROPORTION

FILLED AND A GRADE OF 1:100

(Based on n = 0.013 in Kutter's Formula)

24/16-in.		27/18-in.		30/20-in.		33/22-in.		36/24-in.		39/26-in.		42/28-in.		[illegible]/30-in.		48/32-in.		51/34-in.		54/36-in.		57/38-in.		60/40-in.		63/42-in.		
V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	
2.4	0.2	2.6	0.3	2.9	0.4	3.1	0.5	3.3	0.7	3.4	0.9	3.6	1.1	3.[illegible]	1.3	4.0	1.6	4.2	1.9	4.4	2.2	4.5	2.5	4.7	2.9	4.8	3.1	5
3.6	0.8	3.9	1.2	4.2	1.6	4.6	2.0	4.8	2.6	5.1	3.2	5.4	3.9	[illegible]	4.7	5.9	5.6	6.2	6.6	6.4	7.6	6.7	8.8	6.9	10.1	7.2	11.6	7
4.4	1.8	4.8	2.5	5.2	3.3	5.6	4.3	6.0	5.5	6.3	6.8	6.6	8.3	[illegible].0	10.0	7.3	11.9	7.6	14.1	8.0	16.3	8.2	19.0	8.5	21.8	8.8	24.8	9
4.59	2.32	5.01	3.20	5.43	4.28	5.82	5.56	6.21	7.06	6.59	8.78	6.97	10.78	7.32	13.0	7.66	15.47	8.00	18.24	8.35	21.35	8.67	24.69	8.98	28.35	9.29	32.32	9
5.0	3.3	5.5	4.5	5.9	6.0	6.3	7.8	6.7	9.8	7.1	12.2	7.5	15.0	7.9	18.0	8.3	21.5	8.6	25.3	9.0	29.5	9.3	33.9	9.7	39.1	10.0	44.7	10
5.6	5.1	6.1	7.0	6.4	9.3	7.1	12.1	7.5	15.3	8.0	19.0	8.4	23.3	8.8	28.1	9.2	33.4	9.6	39.3	10.0	45.8	10.4	53.1	10.8	60.9	11.2	69.5	11
.2	7.0	6.8	9.6	7.3	12.9	7.8	16.7	8.3	21.2	8.8	26.3	9.3	32.4	9.8	38.8	10.2	46.6	10.7	54.3	11.1	63.2	11.5	73.2	11.9	84.1	12.3	96.0	12
.3	8.46	6.87	11.68	7.42	15.59	7.94	20.17	8.46	25.57	8.94	31.72	9.41	38.[illegible]4	9.90	46.75	10.35	55.62	10.78	65.44	11.21	76.28	11.65	88.29	12.06	101.27	12.46	115.34	12
.4	9.1	7.0	12.5	7.5	16.7	8.1	21.6	8.6	27.4	9.1	34.0	9.6	41.[illegible]	10.0	50.1	10.5	59.6	11.1	70.3	11.4	81.8	11.8	94.8	12.3	108.8	12.7	124.1	13
.5	11.1	7.1	15.3	7.7	20.5	8.3	26.5	8.8	33.6	9.3	41.7	9.8	51.[illegible]	10.8	61.5	10.8	73.2	11.3	86.3	11.7	100.5	12.2	116.3	12.6	133.5	13.1	152.2	13
.5	12.7	7.1	17.6	7.7	23.4	8.3	30.3	8.8	38.3	9.3	47.6	9.8	[illegible].2	10.8	70.1	10.8	83.5	11.3	98.4	11.7	114.6	12.2	132.7	12.6	152.4	13.1	173.6	13
.92	12.08	6.44	16.65	6.97	22.25	7.47	28.83	7.94	36.48	8.41	45.36	8.86	55.43	9.31	66.80	9.74	79.55	10.16	93.71	10.56	109.22	10.97	126.41	11.37	145.16	11.76	165.45	12

dient of ce is 1:85, then it will be found by the method already frequently owed that a section 63 in. high will be satisfactory, to which a grade of

$$\frac{1}{524.9}\left(\frac{524.9}{150}+\frac{94}{12}-\frac{63}{12}\right)=\frac{1}{86}$$

responds, which agrees closely with the assumed grade. The excess head oducing internal pressure at c is, therefore, $94 - 63 = 31$ in.

If it is desired to avoid this internal pressure, the stretch must be designed the basis of the invert grade, in which case the discharge on a 1 : 100 grade $171.28\sqrt{(150/100)} = 211$ sec.-ft., calling for a 69-in. sewer. In this case re will be a lowering of the surface of the sewage in the top stretch of rer, as shown in the illustration.

7. A storm-water overflow is located as shown in Fig. 11; what is the length its sill if the overflow is assumed to come into operation on a fivefold ution of the dry weather sewage?

The quantities of sewage and the invert grades are indicated in Fig. 11; numbers in parentheses are the quantities during heavy storms, while smaller numbers are the quantities of dry-weather sewage after a five-d dilution. Sewer V has to carry, before the relief outlet comes into action, $3 + 2.90 + 3.99 = 11.02$ cu. ft. per second, consisting of 1.84 cu. ft. of -weather sewage and 9.18 cu. ft. of storm water. This quantity corre-

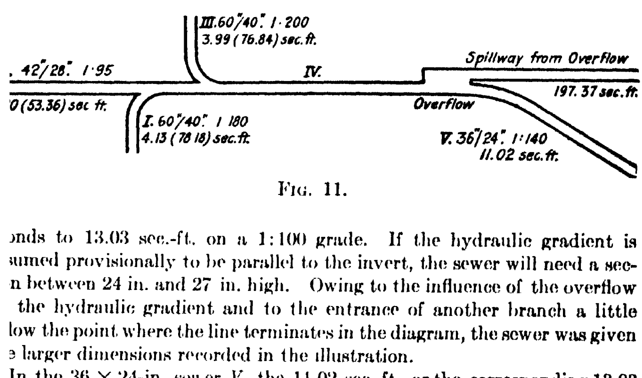


FIG. 11.

onds to 13.03 sec.-ft. on a 1:100 grade. If the hydraulic gradient is sumed provisionally to be parallel to the invert, the sewer will need a sec-n between 24 in. and 27 in. high. Owing to the influence of the overflow the hydraulic gradient and to the entrance of another branch a little low the point where the line terminates in the diagram, the sewer was given e larger dimensions recorded in the illustration.

In the 36×24-in. sewer V, the 11.02 sec.-ft., or the corresponding 13.03 c.-ft. on a 1:100 grade, take up about 18 in. of the height of the section, ich fixes the elevation of the sill of the relief outlet. If internal pressure to be avoided in sewer V, the length of the sill must be such that all the rplus water will flow over it before reaching a depth on the sill of 36 − = 18 in. The length of the sill is determined by the formula already ntioned under "relief outlets." Substituting the quantities of the present se in the formula, gives $b = 197.37 \div 4(18 \div 12)^{1.5} = 26.8$ ft., as the ngth of the sill, which should have a 1:140 slope, corresponding with the ade of the sewer. A shorter sill would be likely to cause an internal pres-

sure in sewer *V*, with a corresponding rising of the hydraulic gradient and an increase in the amount of water flowing through sewer *V*.

Sewer Sections.—The problem of the design of masonry sewers is not solved with the determination of the required carrying capacity, but includes a number of other features which may be of considerable importance.

The most economical shape for the water-way cross-section can only be selected after careful consideration of the special conditions imposed and the relative merits of one type as against another to meet these special conditions. While the circular cross-section has been used for a large number of the masonry sewers constructed in this country there has been an increasing use of other forms such as the horse-shoe, semi-elliptical and rectangular sections. In the older combined sewerage systems constructed previous to 1890, and built for the most part of brick for sizes above 24 in. in diameter, the egg-shaped cross-section was frequently used, but since that time the extended use of separate systems has caused it to decrease in popularity. The old Massachusetts North Metropolitan System was a departure from the practice of the time in that it included such types as the Gothic, catenary and basket handle sections.

The general adoption of concrete for masonry sewers has brought about a more extended preference for the flatter types of inverts on account of their being more easily constructed than the inverts of circular or egg-shaped sections.

Aside from the hydraulic properties, such considerations as the method of construction, character of foundation, available space and stability may be instrumental in determining the best type of sewer section to adopt for a given case.

The selection of the proper thickness of masonry for a given size of sewer, unless determined in the light of experience with similar structures, should be the result of a careful consideration of the forces to be encountered and an analysis of the stresses as determined by the best available methods. This applies particularly to the larger sewers, 6 ft. in diameter and over.

A study of existing sewers is one of the best guides to safe construction although not necessarily the most economical construction. Empirical formulas founded on experience have some value but should not be depended upon without an adequate analytical check.

The proper selection of the materials of construction involves not only a comparison of the cost of one material with that of another but also a consideration of the relative wearing qualities of the materials. This is specially true of the materials used for the lining of the invert.

In some localities the erosion of sewer inverts has been a serious problem responsible for the failure of the entire structure. To resist this wear, a lining of vitrified brick has been found satisfactory.

Sewers are subjected to the action of external forces due to surface loads transmitted through the backfill and to the pressure of the backfilling material itself. Surface loads may be divided into live and dead loads. The former includes such loads as locomotives and other railroad rolling stock, road rollers and heavy vehicles; the latter includes loads from piles of lumber, brick, coal and other materials commonly stored in commercial and manufacturing districts.

With the advent of reinforced concrete has come a greater need for the careful analysis of the masonry section for large sewers. With sewers constructed of brick or plain concrete, the sewer arch if properly designed is subjected only to compressive stresses and depends largely for its stability on the ability of the side walls or abutments to resist the arch thrust. With reinforced concrete, however, the structure as a whole from invert to crown can be designed to resist heavy bending moments and act as a monolith.

The so-called "elastic theory" presents the most rational and practicable means for the analysis of sewer sections. The method of analysis under this theory as described by Turneaure and Maurer in "Principles of Reinforced Concrete Construction" is one of the simplest and best, but for an analysis of the structure as a whole, particularly where the sewer is to be built in compressible soil, the method developed by Prof. A. W. French for the authors is preferable.

Although the previously mentioned aids in design are of the greatest assistance, there must be behind them all sound judgment coming from experience if the best results are to be obtained.

DEPRECIATION OF SEWERS

A sewerage system represents the investment of a large amount of money, usually raised by issuing bonds. If municipalities paid as much attention to financial accounts as private corporations do, the present value of the sewerage and other public works would be ascertained from time to time, just as a railroad company revises its estimates of the value of its physical property. The City of New York, in an endeavor to offset its bonded and other indebtedness by a statement of the actual worth of its property, carried out in 1913 a valuation of the sewers of Manhattan, by methods described in *Engineering News*, Jan. 8, 1914, by Otto Hufeland. The system thus valued was started in the seventeenth century and was built without much public supervision until 1865, when a law came into operation that required the filing of proper plans for sewerage districts. Even after that date it was some years before any comprehensive plans were prepared, and as a result of this condition many of the sewers constructed quite recently were by no means of the capacity or type which the engineers would

select had they been free from the necessity of fitting the new work into the old.

The valuation of the brick and pipe sewers was conducted by different methods. In the case of brick sewers, competent inspectors made a personal investigation of the interior of about 60 sewers, having a total length of about 20 miles. These were divided into four classes, the first including sewers built before 1845, the second those constructed between 1845 and 1855, the third sewers built between 1855 and 1872, and the fourth those constructed between 1872 and 1883. Mr. Hufeland was convinced from close knowledge of the construction and condition of the sewers built after 1883, that it was safe to assume these had not materially deteriorated from their original value. There were a few sewers in this class to which this generalization did not apply, and these were valued independently under known conditions. This opinion of the value of these later sewers was based to a large extent on changes in methods of construction adopted after 1883.

The examination showed that the brick sewers deteriorated in a series of progressive steps. The first sign of service occurred when the cement was found to be partly out of the joint at the water line, a deterioration of about 2 per cent., according to the scale which was adopted after prolonged study. The next type of deterioration was the partial absence of cement above the water line, which was rated as a 6 per cent. injury. Next came a depression of the arch and a slight spreading of the sewer, which was considered a 12 per cent. deterioration; then came, in turn, the appearance of large open joints, rated at 25 per cent.; the existence of loose brick, rated at 47 per cent.; a breaking of the bond of the brickwork, rated at 72 per cent., and finally a distortion of the sides and bottom, with the joints becoming out of line, which was considered complete wreckage of the sewer for serviceable purpose. It was assumed that when the bond of the brickwork became broken, equivalent to a deterioration percentage of 72 on the scale, the sewer was so far gone that it was not economical to attempt to repair it. The condition of the sewer was stated by adding together the percentages of deterioration corresponding to the defects that were observed If all of the defects up to and including the presence of loose brick were observed, then the total of the faults would be $2 + 6 + 12 + 25 + 47 = 92$. This sewer would still be worth repairing, but if the bond of the brickwork was found broken, a 72 per cent. deterioration, the total depreciated value would become 164, when the sewer was considered valueless. Twenty sewers built before 1849 were examined, and nearly every one showed a degree of deterioration exceeding 164, for which reason it was decided that a brick sewer in Manhattan had a useful life of not over 64 years on the average. Sewers built by the methods adopted toward the close of 1883 and subsequently

used have a longer life, of course, a fact which should not be overlooked in any use made of Mr. Hufeland's report.

The first pipe sewers were laid in Manhattan about 1865, and until 1887 they were laid on the earth at the bottom of the trench without any foundation. It required but a slight leak from a joint to wash away the earth enough to permit one end of the pipe to drop so as to cause a serious disturbance of the line. In 1887 the concrete cradle now used in Manhattan was introduced, which resulted in a great improvement in the condition of the pipe sewers in service. Another tendency of the pipe sewers was to break at and above the center, due perhaps to the load imposed on the top or even to some form of disintegration due to this weight, according to Mr. Hufeland. The pipe used in Manhattan were 12, 15 and 18 in. in diameter, and the breaks occurred so much more frequently in the largest size that its use was discontinued in 1887. There were fewer breaks in the 15-in. pipe and still fewer in the 12-in. These pipe lines were examined rather unsatisfactorily by means of reflected lights and calipers, pushed through the pipes by rods. Some information was obtained from the experience of the engineers and workmen engaged in repairing pipe sewers and inserting spurs for house connections; some of the workmen in charge of this labor had been engaged on it for 25 years and were of much help in reaching what was believed to be a fair approximation to the present value of the pipe.

From the information obtained in this way, and a knowledge of the age of the sewers, curves were constructed showing the approximate amount of deterioration of the sewers with their age. One curve answered for brick sewers, but it was considered advisable to use three for pipe sewers, owing to the great difference in the rate of their deterioration with their size. These curves are reproduced in *Engineering News*, but are not given here because they are based on local conditions and poor construction, as already mentioned. In fact, Mr. Hufeland's report everywhere indicates a belief on his part that an investigation of the actual condition of as many sewers as possible should be made before any attempt is made to use this method in appraising the value of a sewerage system.

In this case the results showed that the 2,551,275 ft. of sewers, with 24,383 manholes, cost originally \$22,956,451, and had a value on Dec. 31, 1913, of about \$17,979,123. There were 6,172 catch basins on the system, which were estimated to have a present value of \$685,798, and an original cost of \$923,875. This gives a total cost of the system of \$23,880,326, and a present value of \$18,664,921. This system includes brick sewers of 125 different sizes, 17 sizes of pipe sewers, 23 sizes of wood sewers, about 25 varieties of catch basins, and "all kinds of manholes." The appraisal work lasted over a period of 10 months and cost \$6,053.

CHAPTER III

VELOCITIES AND GRADES

The ratio of the mean to maximum velocity varies with the value of c and with the character of the stream measured. Bazin gives the values recorded in Table 16.

TABLE 16.—VALUES OF THE RATIO OF THE MEAN TO THE MAXIMUM VELOCITY

To be used in obtaining mean velocities from maximum velocities when the value of the coefficient c in the formula $v=c\sqrt{rs}$ is given. (Hering and Trautwine's Translation of Ganguillet and Kutter's "Flow of Water.")

c	$v:v_m$	c	$v:v_m$	c	$v:v_m$	c	$v:v_m$
2	0.06	46	0.64	90	0.78	134	0.84
4	0.13	48	0.65	92	0.78	136	0.84
6	0.19	50	0.66	94	0.79	138	0.84
8	0.24	52	0.67	96	0.79	140	0.84
10	0.29	54	0.68	98	0.79	142	0.85
12	0.32	56	0.69	100	0.80	144	0.85
14	0.36	58	0.69	102	0.80	146	0.85
16	0.39	60	0.70	104	0.80	148	0.85
18	0.42	62	0.71	106	0.81	150	0.85
20	0.44	64	0.72	108	0.81	155	0.86
22	0.46	66	0.72	110	0.81	160	0.86
24	0.48	68	0.73	112	0.81	165	0.87
26	0.50	70	0.73	114	0.82	170	0.87
28	0.52	72	0.74	116	0.82	175	0.88
30	0.54	74	0.74	118	0.82	180	0.88
32	0.56	76	0.75	120	0.82	185	0.88
34	0.57	78	0.75	122	0.83	190	0.88
36	0.59	80	0.76	124	0.83	195	0.89
38	0.60	82	0.76	126	0.83	200	0.89
40	0.61	84	0.77	128	0.83	...	....
42	0.62	86	0.77	130	0.83	...	...
44	0.63	88	0.77	132	0.84	...	...

The ratio of the mean to the maximum surface velocity at a number of places is given in Table 18, from Hering & Trautwine's translation of Ganguillet & Kutter's "Flow of Water."

TABLE 18.—RATIO OF MEAN TO MAXIMUM SURFACE VELOCITIES

Belgrand, for the Seine	(?)0.62
Destrem, for the Neva	0.78
Baumgärtner, for the Garonne	0.80
De Prony, for small wooden channels	0 82
Boileau, for canals	0 82
Cunningham, for the Solani Aqueduct	0 82
Bazin, for small channels	0 83
Swiss Engineers	0 84
Brunnings, for rivers	0 85
Humphreys & Abbot, for the Mississippi (mean)	0 79 to 0 82
Humphreys & Abbot, for the Ohio	0 78 to 0 80
Humphreys & Abbot, for the Yazoo	0 66 to 0 84
Humphreys & Abbot, for the Bayou Plaquemine	0 83 to 0 85
Humphreys & Abbot, for the Bayou La Fourche	0 79 to 0.86

Ratio of the Mean to Center Velocity in Pipes.—In a most valuable article upon "Experiments upon Flow of Water in Pipes" by Williams, Hubbell, and Fenkell (Trans. Am. Soc. C. E., April, 1902) the results of elaborate tests of the relation of the mean velocity to that at the center of a pipe are given, and similar data to those given by the authors were submitted in the discussion which followed the paper. The experiments cover a considerable range of pipes, 2-in. brass tubing, cast-iron pipes of diameters up to 30 in., circular conduits up to 8.75 ft. in diameter, and two rectangular conduits approximately 20×31 in. in section, and indicate that the mean velocity of flow is from 0.80 to 0.85 of the center velocity, the average value found by Williams for cast-iron pipes up to 30 in. in diameter being approximately 0.84. The mean velocity was found to lie at about three-quarters of the radius of the pipe from its center and the velocity at the perimeter of the pipe was found to be approximately one-half of the maximum velocity.

TRANSPORTING POWER OF WATER

The transporting capacity of water, due to its velocity, plays an important part in the disposal of sewage by dilution and diffusion and in preventing clogging of the sewers and local formation of sludge banks, is a result of the settling out of the heavier particles of the sewage. The prevention of clogging of sewers is discussed hereafter under Minimum Grades and Velocities for Sewers. It has been shown that the transporting capacity of water varies as the sixth power of its velocity, so that if the velocity be doubled the transporting capacity is 64 times as great. Therefore any influence which tends to check the velocity at any point immediately results in substantial reduction of its carrying capacity, and

the subsequent deposition of particles which had been carried along readily by the current of greater velocity. The form and adhesive quality of the particles also plays a part in the formation of sludge banks.

Much of the information relating to the transporting capacity of streams is almost valueless, owing to the lack of exact knowledge of the velocity near the bottom of the stream, which, together with the character of the material composing the bottom and the depth and hence the pressure of water upon it, are the most important elements in the problem of erosive action. Freeman has called pointed attention to these facts in his Charles River Dam report, and has cited the opinion of the veteran engineer, Hiram F. Mills, in regard to the misuse of the observations of Dubuat, who made experiments in 1780 upon the capacity of a stream in a wooden trough to move particles on its bottom. All of these observations failed to take into account the varying velocities of flow in any vertical section. Many engineers who have made use of the results of those experiments have failed to recognize this fact, as well as the effect of the character of the material, the coating of slime or colloidal surface which forms upon the bottom and the effect of the pressure upon the material, due to the depth of water. Freeman quotes observations made by Mills and Hale on the Essex Company's canal on the Merrimac River in Lawrence, which were made with sufficient care to be significant, using a current meter to determine the distribution of velocities.

"*At Station No.* 1, middle of west chord of Everett Railroad bridge:

"Banks and bed completely and smoothly covered with fine sand, as per sample, whose mechanical analysis is given in table following. Deposit 8 in. to 12 in. deep. Surface near the bottom marked with little waves of sand ¾ in. high, probably rolled up by the more rapid velocity when emptying canal. Side slopes smooth and free of wave marks. Sand so soft and so like quicksand that one's feet sink into it 3 in. while walking across, or, when standing still for a minute or two, the feet gradually sink into it about 8 to 12 in. This sand plainly is not being scoured, although it is softer than any silt that I have seen uncovered at low tide on the shores of Boston harbor, except perhaps the silty sludge in immediate proximity to certain sewers.

Maximum surface velocity in center found to be	1.3 ft. per second.
Mean velocity of center section	1.0 ft. per second.
Velocity at 3 in. from bottom	0.8 ft. per second.

"This shows that a particularly soft bottom was not eroded by a bottom velocity of about 0.8 ft. per second, and that the condition was one that favored deposits.

"*Station No.* 2, at upstream side of Union Street bridge:

"General appearance the same as at Station No. 1, except that surface of sand in deepest portion of canal is covered by sand waves averaging about 1 in. high, with crests transverse to current, suggesting a rolling along of the

sand grains which perhaps has been induced by the higher velocity from drawing off and refilling the canal a few times very recently, rather than by the ordinary flow. I find, on tramping back and forth over the silt, that it is much more firm than at Station No. 1.

Maximum surface velocity..................	1.9 ft. per second.
Mean velocity of center section..............	1.5 ft. per second.
Velocity at 3 in. from bottom in middle.......	1.2 ft. per second.

"With these velocities silt of this quality is deposited 12 in. deep, and apparently is rolled into waves only by the recent drawing off of canal, since no sand waves are found more than half way up on the sloping sides of canal. The indication is that a bottom velocity of 1.2 ft. per second favors deposit and not scour.

"*Station No.* 3, from same cross-section, but about three-quarters distance up slope from center toward north side and 6 or 8 ft. up from bottom level, where there were no sand waves:

"Deposit 8 in. deep, velocity at about 3 in. from bottom found to average 0.9 ft. per second. Condition here is plainly one of deposit, and not of scour.

"*Station No.* 4, upstream side of Pemberton bridge:

"Upstream from this point the bottom and berms of canal are substantially scoured clean, but a short distance downstream from this point on the northerly edge of berm a deposit begins, and, going downstream, quickly spreads out to 5 ft. in width opposite to the penstocks of the Pemberton Mills, and below this gradually widens out, until at Union Street it covers the entire bed of the canal from north side over to foot of south slope.

"At Pemberton Bridge, where entire bed is scoured clean, there is some irregularity found in the distribution of velocity, but the general average of a dozen or twenty observations ran about as follows:

Mean velocity of entire cross-section.........	2 5 ft. per second.
Velocity 3 in. from bottom at mid-channel ...	1 6 ft. per second.
At 10 ft. from north side.....................	1 5 ft. per second.
In corner next north wall (at deposit)........	0.9 ft. per second.

"The observations at this point show that a velocity of 1.5 ft. per second prevents deposit or produces scour or a rolling along that keeps the bottom clean.

"In general, these north canal observations show that the velocity necessary to prevent deposit or necessary to produce scour of grains of fine river silt and sand of sizes shown by following analysis (Table 19), and forming part of a mass deposited only less than two months before and not compacted by long standing, was not far from 1.3 to 1.5 ft. per second, this velocity being measured at a distance of from 3 in. to 6 in. from bottom.

"These observations thoroughly disprove the oft-quoted, century-old, crude, unreliable observations of Dubuat.

"The boiling and eddying of a current has much to do with its power to transport material in suspension. While this canal has riprap on its banks, its straightness and uniformity of section should offset any greater disturbance than is commonly found in natural streams, and should make the results of general applicability."

TABLE 19.—MECHANICAL ANALYSIS OF AVERAGE SAMPLES OF SAND CAREFULLY COLLECTED FROM WITHIN ONE-QUARTER TO ONE-HALF INCH OF SURFACE AT ABOVE STATIONS, ANALYZED AT LAWRENCE EXPERIMENT STATION, MASSACHUSETTS STATE BOARD OF HEALTH

(From J. R. Freeman's Report on Charles River Dam, 1903, p. 415)

Number of sample	No. 1	No. 2	No. 3
Ten per cent. finer than (diam. in millimeters)	0.12	0.15	0.04
Uniformity coefficient	1.40	1.70	3.60
Finer than 2.04 mm. (per cent. by weight)	100.00	100.00	100.00
Finer than 0.93 mm. (per cent. by weight)	99.60	99.60	100.00
Finer than 0.46 mm. (per cent. by weight)	98.00	97.80	99.00
Finer than 0.316 mm. (per cent. by weight)	95.50	93.40	97.80
Finer than 0.182 mm. (per cent by weight)	66.10	33.10	89.20
Finer than 0.105 mm. (per cent. by weight)	4.30	0.90	32.40
Finer than 0.08 mm. (per cent. by weight)	....	....	19.10
Finer than 0.04 mm. (per cent. by weight)	..	...	9.60
Finer than 0.01 mm. (per cent by weight)			0.90

TABLE 20.—GANGUILLET AND KUTTER UPON THE TRANSPORTING POWER OF WATER

(Hering & Trautwine's Translation of Ganguillet and Kutter's "Flow of Water," p. 124) Col. 2 gives the velocity at the bottom; Col. 3, the mean velocity as figured by Bazin's formula, $v = v_b + 10.9\sqrt{RS}$, in English measure, or an average value of $v = 1.31\ v_b$; Col. 4 contains the maximum surface velocity as figured by Bazin's formula, $v = v_{max} - 25.4\sqrt{RS}$ in English measure, or a mean value of $v = 0.83\ v_{max}$

Nature of material forming bed	Bottom velocity, v_b ft. per sec	Mean velocity, v ft. per sec Bazin	Maximum surface vel., v_{max}, ft per sec. Bazin
(1)	(2)	(3)	(4)
River mud, clay, specific gravity = 2.04	0.25	0.33	0.40
Sand, the size of anise-seed, specific gravity = 2.55	0.35	0.46	0.55
Clay, loam, and fine sand	0.50	0.66	0.79
Sand, the size of peas, specific gravity = 2.55	0.60	0.79	0.95
Common river sand, specific gravity = 3.36	0.70	0.92	1.10
Sand, the size of beans, specific gravity = 2.55	1.07	1.40	1.69
Gravel	2.00	2.62	3.15
Round pebbles, 1 in. diam., specific gravity = 2.61	2.13	2.79	3.36
Coarse gravel, small cobblestones	3.00	3.93	4.73
Angular stones, flint, egg size, spec. gravity = 2.25	3.23	4.23	5.09
Angular broken stone	4.00	5.24	6.30
Soft slate, shingle	5.00	6.55	7.86
Stratified rock	6.00	7.86	9.43
Hard rock	10.00	13.12	15.75

Bazalgette found the following velocities in feet per second were necessary to move the bodies described: fine clay, 0.25; sand, 0.50; coarse sand, 0.66; fine gravel, 1.00; pebbles 1 in. diameter, 2.00; stones of egg size, 3.00.

Blackwell showed by experiments made for the British Metropolitan Drainage Commission that the specific gravity has a marked effect upon the velocities necessary to move bodies, as given in Table 21.

TABLE 21.—EFFECT OF SPECIFIC GRAVITY ON SUSCEPTIBILITY TO VELOCITY OF WATER

(Hering and Trautwine's Translation Ganguillet and Kutter's "Flow of Water," p. 125)

Nature of bodies	Specific gravity	Velocity in feet per second
Coal....	1.26	1.25 to 1 50
Coal.....	1.33	1.50 to 1 75
Brickbat...	2.00	1.75 to 2 00
Piece of chalk	2.05	
Oolite stone	2.17	
Brickbat..	2.12	2.00 to 2.25
Piece of granite	2.66	
Brickbat....	2.18	2.25 to 2.50
Piece of chalk....... ..	2.17	
Piece of flint	2.66	2.50 to 2.75
Piece of limestone .. .	3.00	

Note that in both of the above quotations there is no discrimination between surface, mean, or bottom velocities.

The Metropolitan Sewerage Commission of New York, 1910, assumed the velocities given in Table 22 to be necessary to move solid particles.

TABLE 22.—CURRENTS NECESSARY TO MOVE SOLIDS

(Metropolitan Sewerage Commission, New York)

Kind of material	Velocity required to move on bottom	
	Feet per second	Miles per hour
Fine clay and silt..................	0.25	about 1/6
Fine sand.........................	0.50	about 1/3
Pebbles half inch in diameter........	1.0	about 2/3
Pebbles 1 in. in diameter	2.0	about 1 1/3

In general it is found that a mean velocity of 1 ft. per second, or thereabouts, is sufficient to prevent serious deposition of sewage upon tidal flats, if the sewage is reasonably comminuted.

The interesting experiments both of Professors Adeney and Letts of the Royal Commission, and Clark of the Massachusetts State Board of Health (the latter made in connection with Freeman's Report upon the Charles River Dam) conclusively point to the fact that the polluting organic matter is precipitated very much more rapidly in salt water than in fresh. The danger of formation of sludge banks from the discharge of a given quantity of sewage into a body of salt water, is greater therefore than in the case of a like body of fresh water. This is not a phenomenon depending upon the transporting power of flowing water, however, although it might be confused with it.

EROSION OF SEWER INVERTS

The erosive effect of sewage upon sewer inverts of different kinds is unimportant in the case of the separate system unless there be chance for the entry of sand, gravel or other silicious material. In the combined system, however, which has to deal with silicious material as well as with rain water and sewage, the effect may be important. The rapidity of the erosive action will depend not only upon the velocity of flow, but also upon the character of the material transported, arenaceous material being much more destructive in its influence than argillaceous or limestone, on account of its greater hardness. Vitrified sewer pipe is resistant to erosion and has been laid successfully upon very steep grades. In large combined sewers, it has generally been customary to line concrete or brick sewers, in the invert at least, with vitrified brick, where the velocity of flow is in excess of 8 ft. per second, although some engineers have used as low a limit as 4 ft. per second. Wrought-iron or steel inverts have been used in some very steep sewer outfalls; in others, a depressed channel has been made in the main sewer, lined with split tile, vitrified brick or steel, large enough to carry the dry weather flow, the remainder of the invert being formed in concrete or lined with vitrified brick so that in case of need of repairs, it should be possible to get into the sewer during the dry weather season to make the repairs without interruption of service.

It seems likely, in view of the accumulating favorable experience with concrete inverts, in irrigation as well as sewerage works, that concrete inverts may be used without a lining of vitrified brick for higher velocities than heretofore customary, except in those cases where the sewage is exceedingly stale, or impregnated with deleterious chemicals,the sewer badly ventilated, and the materials transported very hard in character.

The Metropolitan Sewerage Commission of New York reported in 1910, with reference to erosion in the outlets of the sewers inspected, that few cases were found where the bricks of the inverts were actually worn away. In a few places in the upper west side of Manhattan, the upstream edges of the bricks were rounded off as a result of the high velocity of sewage. In a large number of the sewers the mortar was worn from the joints in the brickwork of the invert. Sometimes the mortar has been worn away only to a slight depth while at other places it has been cut out by the sewage to the full depth of the brick.

In combined sewers at St. Louis, with grades ranging from 0.2 to 2 per cent., averaging about 0.5 per cent. for sewers more than 5 ft. in diameter, and about 1 per cent. for those of smaller sections, vitrified clay pipes were stated by E. A. Hermann, in *Eng. News*, Feb. 4, 1904, to show no appreciable wear after about 35 years use, vitrified brick inverts to show no appreciable wear after about 12 years, and inverts of

ordinary sewer brick to show some wear after about 3 years service and from 2 to 4 in. wear after a use of 30 years.

MINIMUM GRADES AND VELOCITIES

The transporting capacity of water is important on account of its bearing upon the possible clogging of sewers. The actual conditions of flow in the sewers must also be clearly borne in mind.

As will appear in the diagrams showing the hydraulic elements of various sewer sections, the velocity of flow in any sewer laid upon a given grade varies markedly with the depth of sewage flowing. Obviously, the quantity flowing also varies greatly, at different hours of the day, as discussed in the chapter on the quantity and variation in flow of sewage. At times of low flow of sewage, the velocity will be so low that the stream will be able to transport only the finely comminuted suspended matter; the paper, street washings and other foreign matter contained in the water will temporarily find lodgment upon the bottom and sides of the sewer. If the foreign matter is sufficient in amount to cause clogging, pooling of the sewage behind the obstruction will result until the volume and pressure of liquid are sufficient to break through the obstruction and develop a velocity which will again pick up the arrested material and transport it. Owing to the grease contained in the sewage and the conditions of flow, however, some of the material may not be picked up again at the same velocity as that at which it was transported when in a suspended condition, and obstructions are thus formed and gradually built up to a point where sufficient velocity is developed to maintain a channel through the sewer.

From the point of view of operation, it is important that the minimum velocities assumed in the design of the sewer, when flowing one-half, two-thirds, or full, as the case may be, shall be adequate to keep it thoroughly flushed. In general, it has been found that a mean velocity

> of 2-1/2 ft. per second will ordinarily prevent deposits in sewers built upon the combined system and,
>
> 2 ft. per second will ordinarily prevent deposits in sewers built upon the separate system.

It is desirable, however, that a mean velocity of 3 ft. per second, or more, shall be obtained where possible, and this minimum limit should not be lowered in the case of inverted siphons under any ordinary conditions. While lower minimum velocities have been used in different places, they have often been accompanied with more or less expense for removing sediment by which the sewers might in time become clogged, and such work is expensive. Slopes giving as low velocity as 1.5 ft. per second have been successfully used, where imperative, in sewers built

upon the separate system, but they are undesirable and are likely to lead to greater cost in maintenance.

In general, the minimum grades given in Table 23 for small pipe sewers laid upon the separate system have been found safe though steeper grades are always desirable. These grades are the least ordinarily permitted by the New Jersey State Board of Health. In its 1913 regulations governing the submission of designs, it stated:

"The sewers should have a capacity when flowing half full sufficient to carry twice the future average flow 25 years hence, plus a sufficient allowance for ground-water infiltration. When grades lower than those given are used, an explanation and reasons for the use of such grades should be included in the engineer's report."

TABLE 23.—MINIMUM GRADES IN SEPARATE SEWERS; FOR 2-FT. VELOCITIES[1]

Diameter, inches	Minimum fall in feet per 100 ft.
4	1.2
6	0.6
8	0.4
10	0.29
12	0.22
15	0.15
18	0.12
20	0.10
24	0.08

[1] $n = .013$

The velocity of flow and capacities of the sewers are determined in Manhattan, Brooklyn, The Bronx, Queens, Richmond, Newark, the East Jersey Joint Outlet Sewer, Elizabeth, Jersey City and Hackensack by the use of Kutter's formula. In Kutter's formula the value of n, which takes into account the roughness of the interior surface of the sewer is assumed for pipe sewers to be, in all the cities, 0.013; for brick sewers in Manhattan, Richmond and Jersey City it is taken as 0.013 and in Brooklyn and The Bronx and Queens, Richmond, Newark and Hackensack at 0.015. For concrete in Manhattan 0.011 is used; in Brooklyn, Queens, Newark, 0.015; in The Bronx, 0.014, and in Richmond, 0.011 for smooth finished concrete. (Report of Metropolitan Sewerage Commission of New York, 1910, p. 90.)

Imhoff has used successfully minimum velocities of 2.3 ft. per second in masonry-lined open channels built by him in Germany, but these channels carry the effluent from the Imhoff or Emscher tanks and not the raw sewage of the communities through which they pass.

The main intercepting sewer at Columbus, Ohio, 2½ to 6 ft. in diameter, laid upon grades of 0.61 ft. per 1000 for the small section to 1.94 ft. for a 36-in. section, giving velocities from a minimum of 1.72 ft. to a

maximum of 3.6 ft. per second (assuming the sewer to flow full and n to equal 0.015), has given considerable trouble from the collection of large quantities of sediment. The cost of removing this sediment is reported to have been approximately $1.78 per cubic yard. It should be stated, however, that an unusually large amount of sediment entered this interceptor on account of the defective design of the connections of the lateral sewers with it, and the discharge of tar into it from a gas plant. (*Trans. Am. Soc. C. E.*, vol. 67, pp. 326–327, 433–434.)

Part of the Boston Main Drainage Works consists of a tunnel 7.5 ft. in internal diameter and 7166 ft. long. The ordinary velocity through this tunnel at the inception of the works was about 1 ft. per second. To ascertain the extent of deposits under these conditions, water was pumped in at one end and the difference in level at the two ends was noted for the purpose of figuring the value of c in $v=c\sqrt{rs}$. It was assumed that when this value approximated 137 it would indicate that there were no deposits. The results of these experiments are given in Table 24. These figures indicate that deposits occurred with a velocity of approximately 1 ft. per second and did not occur with a velocity of approximately 4 ft. per second. (Boston Main Drainage Report, 1885.)

TABLE 24.—EXPERIMENTS AT BOSTON TO DETERMINE VELOCITIES AT WHICH DEPOSITS OCCUR

No. of experiment	Mean velocity, ft. per second	Value of c in $v=c\sqrt{rs}$	Liquid flowing
1	0.929	79.95	Sewage
2	0.998	82.00	Sewage
3	3.988	129.05	20 per cent. to 25 per cent. sewage, 75 per cent. to 80 per cent. salt water.
4	0.965	109.66	Sewage
5	3.929	120.67	20 per cent. to 25 per cent. sewage, 75 per cent. to 80 per cent salt water.
6	3.897	146.31	Ditto
7	4.062	146.64	Ditto

H. P. Eddy, *Jour. Assoc. Eng. Soc.*, 1904, p. 235, gives his observations upon certain sewers in Worcester, Mass., in Table 25.

The Metropolitan Sewerage Commission of New York, in its sixth Preliminary Report, 1913, fixed from 2 to 5 ft. per second as a suitable range of velocities to prevent deposit from screened sewage from which the grit has first been removed, in a proposed siphon 2300 ft. long from 8 to 9 ft. in diameter, to be laid 110 ft. below the surface of mean low water to carry the sewage (99,000,000 gal. a day in 1915) from Manhattan Island to Brooklyn beneath the lower East River.

TABLE 25.—OBSERVATIONS AT WORCESTER OF VELOCITIES AT WHICH DEPOSITS DO AND DO NOT OCCUR

Street	Kind of sewer	Size, inches	Approx. mean velocity, ft. per second	Shape	Remarks upon deposit
Pink......	Storm . .	24x36	2.06	Egg....	Deposit occurs.
Pink... .	Storm . . .	18	1.47	Egg...	Deposit occurs.
Pink.....	Storm . .	18	1.46	Egg ..	Deposit occurs.
Pink.......	Storm. . .	18	1.17	Egg...	Deposit occurs.
Pink......	Storm..	18	2.86	Egg....	Deposit occurs.
Pink..	Storm .	18	1.13	Egg...	Deposit occurs.
Pink..	Storm	18	2.14	Egg..	Deposit occurs.
Highland . .	Storm..	18	3.74	Egg....	No deposit.
Highland	Storm	18	3.02	Egg..	No deposit.
Highland	Storm . .	12	2.26	Round.	No deposit.
North . ..	Combined .	22x33	1.99	Egg....	Deposit occurs.
North .. .	Combined .	18	1.94	Egg....	No deposit.
North.	Combined. .	18	2.25	Egg...	No deposit.
North... . .	Combined...	18	1.72	Egg...	No deposit.
North	Combined .	18	2.01	Egg .	No deposit.
North. . . .	Combined	15	2.56	Egg.. .	No deposit.
North.. . . .	Combined.	15	1.54	Egg..	No deposit.
North.... . .	Combined .	12	4.63	Round..	No deposit.
North..	Combined	12	6.97	Round .	No deposit.

The Pink and Highland street sewers form a single line, beginning with a 12-in. round section. The figures begin at the bottom and should be read upward. There was no trouble until the velocity dropped to 2.14 ft. per second. The reason that trouble is experienced where the velocity should theoretically be 2.86 ft per second probably lies in the flat grades on each side of it. In the case of the North street sewer, no trouble is experienced until the lower end is reached, where for about 450 ft. the velocity falls to 1.99 ft. per second. This is not so low as the velocities at several other places, but each of the latter is preceded by at least one section which has a good velocity.

In St. Louis sewer designing under W. W. Horner (*Eng. News*, Sept. 5, 1912) the curves were sometimes compensated by the Markmann formula (*Eng. News*, Sept. 29, 1910). This formula is:

$$S_c = S + v^3/2gr^2 \qquad (1)$$

where S_c is the grade or fall per foot on the curve, equivalent to a grade S on the tangent, v the velocity due to the grade S, r the radius of the curve in feet, and g the acceleration of gravity. In applying the formula it is necessary for the velocity to be known, and this can only be obtained from the finished computations. This involves a system of trials, and consequently the custom in St. Louis was to make the compensation and consequent modification of the tangent grades a part of the records. The runoff was determined by the method described in Chapter VIII on estimating runoff, and a preliminary grade was plotted. If the section under investigation consisted of a tangent of length l and a curve of length l_c, the actual grades would be $S_c l_c$ and Sl and the total drop in grade, F, would be

$$F = S_c l_c + Sl \qquad (2)$$

In practice, a grade was assumed somewhat less than the preliminary grade and from this and the required capacity the velocity was quickly determined from diagrams. These values were substituted in Eq. 1 and a value of S_c obtained. The results were checked by substitution in Eq. 2.

Engineers' Opinions Regarding Minimum Grades.—The following opinions as to safe practice in selecting minimum grades were furnished, in 1913, to the authors by the engineers whose names are given.

James N. Hazlehurst stated that his practice had been largely in connection with sewer systems in the southeastern coast states, where there is much silt and running sand. Minimum grades were absolutely necessary to accomplish anything and he generally used grades lower than those recommended in text-books. The minimum grade for each size of pipe sewer, which he ordinarily permitted, was: 6-in. sewer, 0.33 per cent.; 8-in., 0.25; 10-in., 0.20; 12-in., 0.17; 15-in., 0.15; 18-in., 0.12; 20-in., 0.10; 24-in., 0.08. When sewers were properly constructed he reported that he knew of no trouble from deposits when the grades were not lower than those stated. In Waycross, Ga., there were 8-in. pipe sewers on grades as flat as 0.24 per cent., which operated without giving trouble; a few grades which were as flat as 0.10 per cent., however, were clogged from time to time and had to be rodded out.

Charles B. Burdick stated that it was the practice of Alvord and Burdick to secure grades that would give a velocity of 2 ft. per second in separate sewers flowing full or half full, and to reduce the grade to 1½ ft. per second, if necessary. Even on such grades they used flush tanks at the summits of the laterals, and if these velocities could not be obtained, special flush tanks were usually installed. On combined sewers they endeavored to secure 3 ft. velocity, but reduced it to 2 ft. if necessary. They stated: "It is our practice to get all the grade we can at reasonable expense, and if it is impossible through physical conditions or cost to get the grade desired, we usually install some means for flushing, with the idea of removing deposits. We have in several cases installed a specially capacious flush tank at the head of a main where an unusually flat grade is used, these especially flat grades coming more commonly on mains than laterals."

George G. Earl stated that the standard minimum grades for sewers in New Orleans, given in Table 26, were occasionally exceeded, because it has been necessary in some cases to lay considerable 8-in. pipe on grades as low as 0.25 per cent. The aim is to have a velocity of 2 ft. per second in a half-full 8-in. pipe, and a slightly increasing velocity in half-full sewers as the size increases. The sewers are of terra cotta pipe up to 30-in. diameter, and either brick or concrete in larger sizes. Some of those over 30 in. in size are semi-elliptical in section, but on account of constant leakage the volume of flow is sufficient at all times to render circular sections fairly satisfactory.

In the drainage system at New Orleans, better bottom grades are usually obtained than in the sewers. The main drains have a V-shaped bottom, with the bottom slopes about 1:4; they are 4 to 25 ft. wide, with good bottom gradients which give velocities of 5 to 10 ft. per second when running full. The laterals enter them at the top of the bottom slope, and thus have the maximum grade practicable. Mr. Earl stated that the drainage system, particularly the terra-cotta pipe laterals from 10 to 30 in. in diameter, receive street washings and sweepings in dry weather when the flow is inadequate to remove them, and consequently a good deal of flushing and cleaning is required on account of these dry-weather accumulations.

TABLE 26.—MINIMUM GRADES ON NEW ORLEANS SEWERS

Diameter, inches	Slope, per cent.	Diameter, inches	Slope, per cent.	Diameter, inches	Slope, per cent.
8	0.33	27	0.100	48	0.062
10	0.25	30	0.091	51	0.059
12	0.21	33	0.083	54	0.056
15	0.167	36	0.083	57	0.053
18	0.133	39	0.077	60	0.050
21	0.114	42	0.071	63	0.050
24	0.100	45	0.067	66	0.050

George W. Fuller stated that his drafting-room practice for separate pipe sewers was based on a 2-ft. velocity when half-full, with a coefficient of roughness, n, of 0.013. This coefficient is also used for concrete sewers 24 in. in diameter and over, and 0.015 is used for brick sewers. Rather than go to the expense of pumping where the grades tend to make it necessary, the slopes giving the velocities mentioned are sometimes flattened. This is done, however, only after a careful examination of local conditions on the ground, and is not normal office practice. For instance, at Vincennes, Ind., in a sewerage system designed two years ago, Mr. Fuller made use of grades of 3 ft. per thousand with 8-in. pipe and in some cases a grade of only 2.5 ft. per thousand was used. J. R. McClintock reported subsequently for Mr. Fuller that an examination of the Englewood, N. J., sewerage system revealed a number of sewers with very low grades, which were apparently quite satisfactory. Six-inch sewers were flowing freely with grades as low as 3.5 ft. per thousand, and there were cases of 12-in. pipe with a grade of about 1 ft. per thousand, and 8-in. pipe with grades of 1 ft., 1½ ft. and 2 ft. per thousand in satisfactory operation. There were other sections on this same system, however, where sewers with grades apparently as low were partly clogged.

Mr. Fuller stated that in the case of separate sewers he was of the opinion that the depositing velocities would not have appreciable significance if substantially every day there were periods when the velocity approached 28 in. per second or more. He stated carefully to clients

that where the slopes of sewers, more than two or three blocks removed from flush tanks at the head of a line, showed a velocity of less than 20 in. per second, care should be taken to flush the sewers either by a hose or some equivalent. In the case of combined sewers he endeavored to secure a nominal minimum velocity of 2-1/2 ft. per second. In practically every case where he has had occasion to study in detail the condition of intercepting lines, a heavy grit has been found deposited in them. If these deposits were not removed, they apparently decomposed and became more or less cemented by ferrous sulphide. The result was that scouring velocity applicable to ordinary street wash would not longer suffice. This he found quite marked in Elizabeth, N. J., although the data are too meager to find place in a record of accurate information.

C. E. Grunsky stated that the minimum grades in Californian cities, reported to him by the engineers of the places named, were as given in Table 27. The city engineer of Stockton said that the grades in that

TABLE 27.—MINIMUM GRADES IN CALIFORNIA CITIES

Size, inches	Stockton, per cent.	Fresno, per cent	Modesto, per cent.	Visalia, per cent.	Sacramento, per cent.
6	0.2	0.15	0.16	0.3	0.25
8	0.143	0.1	0.16	0.24	0.2
10	0.139	0.1	0.2	0.143	0.16
12	0.1	0.1	0.152	0.143	0.12
15	......	....	0.09	...	....
18		.	. .	0.1	. . . ..

city have given no trouble during the 25 years they have been in service; these sewers carry only sewage, rain water being excluded. Once in a great while they have had some trouble from deposits at Fresno, due to sluggish flow, according to the city engineer. The city engineer of Visalia stated that he had made float measurements of the sewers and found that the actual minimum velocity when they were running one-third to one-half full, was 1.1 ft. per second in 10-in. sewers, and a velocity of 1.75 ft. per second was observed in an 18-in. sewer half full. The light grades caused no trouble in that city. The city engineer of Sacramento stated that the depth of flow in the sewers of his city did not average one-fourth of their diameters; in no case had there been any offensive deposits.

T. Chalkley Hatton in experiments with two 24-in. sewers discharging creek water carrying considerable clay, the grade being 0.077 per cent., found no appreciable sediment with the following depths in inches and velocities in feet per second:

Depth	5	12	12
Velocity	1.21	2.35	1.70

Alexander Potter stated that his general practice was to lay all sewers at a grade giving a velocity, when flowing half full, of at least 2 ft. per

second and preferably 2-1/2 ft. With grades giving velocities much less than 2 ft. per second when flowing half-full, flushing and frequent cleaning are necessary. In order to avoid pumping or costly construction, however, Mr. Potter has used very flat grades at times. At Harrison, N. Y., about 5000 ft. of 20-in. sewer was laid with a fall of only 1.1 ft. per thousand. As the average flow will never more than quarter fill the pipe, arrangement has been made to flush it automatically once a day. At Kingsville, Tex., in order to avoid pumping, sewers flushed automatically once a day have been laid on grades as low as 0.1 per cent for 18-in. and 15-in.; 0.15 per cent. for 12-in., 0.2 per cent. for 10-in., and 0.33 per cent. for 8-in. In the southern part of Texas where the land is very flat many 8-in. sewers have been laid with a fall of only 2 ft. per thousand. In Corpus Christi, Tex., Mr. Potter found that practically all 8-in. laterals had been laid with a minimum grade of 0.2 per cent., and were kept clean by frequent flushing.

EXAMINATION OF SEWER DESIGN WITH REFERENCE TO MINIMUM FLOW CONDITIONS

Economic considerations generally require the construction of main or intercepting sewers to meet future rather than present needs. The length of the period to be cared for will be determined by the attendant circumstances, but in general these sewers are designed to meet the needs of a period of from thirty to fifty years. As a result of this, the flow in the sewer for a long period of time will be much below the normal conditions for which it is designed.

It is necessary, therefore, after designing a sewer for a given service in the future, to consider the actual conditions of operation likely to arise under dry weather or minimum flow during the first few years after its construction, in order to make certain that the velocities will not be so low, for significant periods of time, as to cause serious deposits in the sewer, the removal of which would involve unwarranted cost. The construction of a sewer to serve for the long periods assumed above would be unwarranted if the cost thus resulting should exceed the cost of building a smaller sewer in the first instance, to serve for a shorter period of time and until the anticipated growth had developed in some degree, and of then building a second sewer to take care of the additional sewage flow resulting from the added growth. While the latter plan would involve greater first cost of construction, enough might be saved in fixed charges and in the cost of operation, in the early years of the use of the sewer, to more than cover this increased cost.

An example of such a computation is shown in Table 28. It will be noted in Column 14 that the estimated velocities under full flow, thirty years after the construction of the sewer, range from 2.4 to 3.1 ft. per

TABLE 28.—SIZE OF PROPOSED ACUSHNET INTERCEPTING SEWER, NEW BEDFORD, MASS., BASED UPON METCALF AND EDDY'S ESTIMATE OF REQUIRED CAPACITIES

Sewer at	Station or distance from point of discharge feet	Length of interval in feet	Population tributary in 1910	Population tributary in 1940	Resulting Q in mgd. Dry weather 125 gpcd. = mgd 1910	Dry weather 1940	Storm flow 400 gpcd. = mgd. 1910	Storm flow 1940	Sewer Dia. inches	Slope per 1000 ft.	Fall in feet	Flowing full (1940) Capacity cfps.	Flowing full (1940) Velocity feet per second	Dry weather flow (1910) cfps.	Depth in feet	Velocity, feet per second	Assumed elevation of invert
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
North End	37,300	......	242	......	0 03	......	0.1	....	...	...	...	...	...	...	...	...	10.68
Nash Road	36,000	1,300	5,072	15,000	0.63	1.9	2.0	6.0	36	0.7	0.91	(17.5)	2.5	......	......	......	9.77
Coffin Avenue ..	33,800	2,200	10,308	28,120	1.29	3.5	4.1	11.2	42	0.5	1.10	9.3 (22 5)	2.2 (2.4)	1.0	0.5	1.1	9.27 8.17
Sawyer Street...	32,200	1,600	21,284	65,240	2.66	8.2	8.5	26.1	48	0.50	0.80	27.2	2 9 (2 6)	2.0	0.6	1.4	7.67 6.87
Acushnet Avenue.	29,100	3,100	30,780	76,800	3.85	9.6	12.3	30.7	54	0.50	1.55	40.4 (45)	3 1 (2.8)	4.1	1.0	1.7	6 37 4.82
Willis Street.....	25,400	3,700	49,056	110,980	6.14	13.9	19.6	44.4	60	0.40	1.48	47.5 (52)	3.0 (2.7)	6.0	1.2	1.7	4.32 2.84
Elm Street......	22,300	3,100	54,976	123,060	6.87	15.4	22.0	49.2	66	0.40	1.24	68.6	2.9	9.5	1.4	2.0	2.34 +6.10
Howland Street.	18,600	3,700	73,504	157,140	9.19	19.6	29.4	62.9	72	0.33	1.22	76.1	2.7	10.6	1.5	1.9	+0.60 −0.62
Rivet Street	16,400	2,200	98,252	202,140	12.3	25 2	39.3	80.8	78	0.33	0.73	97.2	2.9	14.3	1.69	2.0	−1.12 −1.85 −2.35
Cove Road	13,600	2,800	105,500	220,695	13.2	27.6	42.2	88 3	90[1] 84[2]	0 30	0.84	132.	3.0	19.0	1.91	2.1	−3.19
Screening Station.	7,900	5,700	......	......	...	...	......	......	90[1] 84[2]	0 30	1.71	132. (85 5 mgd.)	3.0+	20.8	.....	....	−4.9
Shore Point.....	3,200	4,700	......	......	...	...	...	...	90[1] 84[2]	....	...	...	....	......	...	......	......
HarborDischarge	0	3,200	...	......	...	...	...	...	.	...	...	....	.....	...	.....	......	......
Total.......	......	37,300	...	.....	...	...	...	.		.	...	...	...	...	..	....	....

[1] Horseshoe type. [2] Circular type. gpcd = gallons per capita daily: mgd = million gallons daily.

Note.—Amounts in the parentheses indicate actual amounts corresponding to selected sizes of sewers, where other figures denote required figures.

second, whereas the velocities for the anticipated dry weather flow at the beginning of the period range, in general, from 1.7 ft. to 2.1 ft. per second, though at the head of the sewer, velocities as low as 1.1 ft. per second were anticipated.

It is desirable that the sewer sections and slopes should be so designed that the velocity of flow will increase progressively, or at least be maintained, in passing from the inlets to the outlet of the sewer, so that solids washed into the sewer and picked up and transported by the flowing stream may be carried through and out of the sewer, and not be dropped at some point owing to a decrease in velocity.

It is obvious that the velocity of flow is but one of many factors involved which must be given consideration in such an economic study; nevertheless, it is one which should be carefully weighed and not lost sight of.

Velocity in Submerged Sewers.—Computations relative to the dry weather and minimum flows in submerged sewers, particularly such as sewer outfalls, must also be made, for here the conditions tending toward clogging of the sewer are particularly aggravated. Unless grit chambers or other devices for removing the heavy mineral matter are provided, the danger of clogging may be serious. This danger arises from the fact that where the pipe is submerged, flow takes place in the entire cross-section and with a given rate of flow the velocity may thus be reduced to exceedingly small limits.

Fortunately, however, the matter in suspension, if of organic character only, tends to remain in a semi-flocculent condition, buoyed up in part by its low specific gravity and in part by the rising bubbles of gas formed by its putrefaction, so that if the sewer does discharge under substantial velocity from time to time during the day, or even at longer intervals, the flow may maintain the sewer reasonably free from clogging deposit.

If such outfalls are into salt water, the effect of the difference in specific gravity of the two liquids is to be borne in mind as well as the fact that the salt water tends to precipitate the suspended organic matter more quickly than does fresh water.

Flush Tanks for Dead Ends.—The difficulty of obtaining adequate velocities of flow in the extremities of the sewer pipe system, where the grades are, of necessity, very flat, is met by the use of flush tanks or by flushing the sewers periodically in other ways. Such devices though necessary under certain conditions are, at best, a source of annoyance and expense on account of the difficulty of making them operate regularly automatically and of the expense of furnishing water for the purpose of flushing. Moreover, the action produced in the sewer by the discharge from the flush tank is a purely local one as the influence of the flood wave is felt for but a short time and to a comparatively short distance, as explained in Chapter XV.

HYDRAULIC ELEMENTS OF SOME STANDARD SEWER SECTIONS

In Figs. 38 to 41, inclusive, are given the hydraulic elements of certain standard sewer sections, which have been figured by the application of the principles outlined in this chapter. The computation of the elements of sewer sections other than the circular is a rather long process and can be considerably lightened by using a planimeter where extreme accuracy is not desired. The hydraulic elements of other sections are given in Chapter XI on the design of masonry sewers.

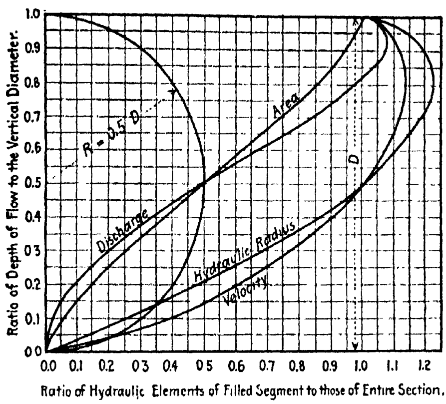


FIG. 38.—Hydraulic elements of circular section by Kutter's formula. $n = 0.013$; $s = 0.0003$; $D = 7\frac{1}{2}$ ft Area $= 0.785D^2$, Wetted Perimeter $= 3.1416D$; Hydraulic Radius $= 0.250D$.

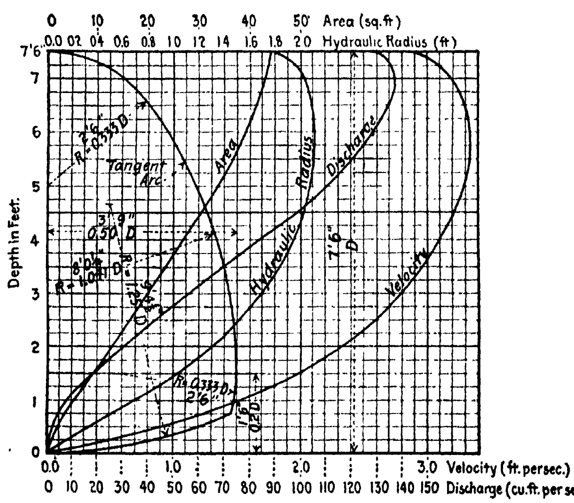


FIG. 39.—Hydraulic elements of semi-elliptic section by Kutter's formula. $n = 0.013$; $s = 0.0003$.

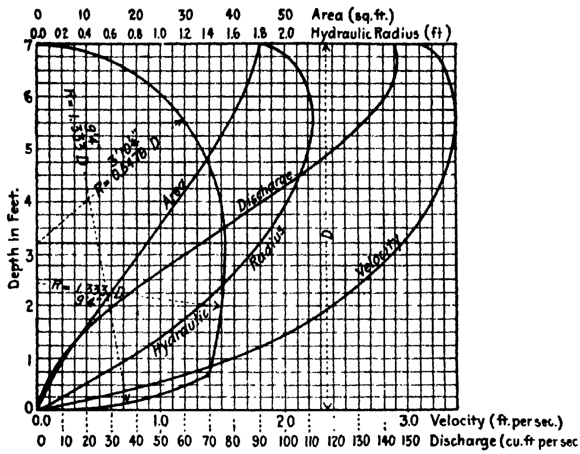


FIG. 40—Hydraulic elements of horseshoe section, Wachusett type, by Kutter's formula.
$n = 0.013$; $s = 0.0003$.

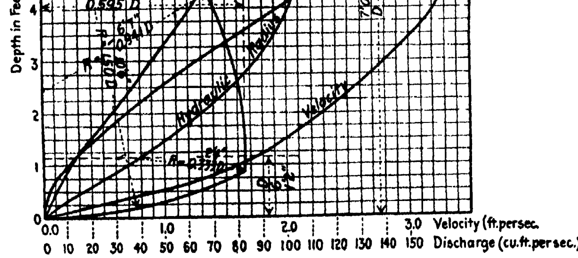


FIG. 41.—Hydraulic elements of special semi-elliptical section by Kutter's formula.
$n = 0.013$; $s = 0.0003$.

CHAPTER IV

MEASUREMENT OF FLOWING WATER

The discharge from sewers or drains may be measured by the following different methods, the choice depending upon the conditions found:

1. By weighing the discharge for a given period of time in tanks or other receptacles.
2. By measuring the discharge for a given period of time in tanks or other receptacles, the contents of which can be accurately gaged.
3. By standard orifices.
4. By standard weirs of the rectangular, triangular or trapezoidal form.
5. By Venturi meter.
6. By current meter.
7. By float measurements.
8. By observing the depth of flow at two adjacent points, when a fair condition of hydraulic equilibrium has been reached, and figuring the discharge under the given hydraulic elements, depth, slope and area of cross-section, by suitable formulas.

The use of the Pitot tube, which has proved so useful in clear water pipe flow gagings, is impracticable in sewer gagings, on account of the suspended matter contained in the sewage. Nozzles are also of little use on account of lack of pressure.

In the following paragraphs will be found a brief discussion, with accompanying formulas, tables and curves for convenience in computation, relating to measurement by orifice, weir, Venturi meter, float, or current meter. The application of the formulas already discussed to the determination of the quantity of sewage flowing in any sewer requires no explanation, the method being at best an approximation dependent upon the steadiness of the flow at the time of observation and the precision with which the coefficient of roughness is estimated for the existing conditions. Nevertheless, the last method is the one most commonly used in ordinary sewerage work and is sufficient for the needs of the superintendent of sewers in his everyday practice. For special investigations, one of the other methods suggested must be used. The method selected will depend upon the facilities at hand, the degree of precision required and the conditions under which the sewer was built and is operating.

For a further discussion upon the measurement of flow in sewers, reference may be had to Chapters VI and IX.

Discharge through Orifices.—In accordance with Torricelli's theorem, that the velocity of flow through the orifice is equal to the velocity acquired by a freely falling body in a space corresponding to the head over the orifice, the discharge through an orifice is as follows:

$Q = cav = ca\sqrt{2gh}$, in which
Q = quantity, in cubic feet per second
c = coefficient of discharge
a = net area of orifice, in square feet
v = velocity, in feet per second
h = head, in feet, from center of orifice to surface of water
g = acceleration of gravity = 32.16

The coefficient c is required by reason of the fact that the cross-section of the jet, at a point a short distance outside the orifice, has generally a somewhat smaller area than that of the orifice itself, the reduction in area depending upon the character of the orifice. When the edge of the orifice is sharp so that the water does not adhere to the orifice, the coefficient is at a minimum or the reduction in area is at a maximum. When, on the other hand, the orifice is shaped to a bell-mouth, the coefficient is at a maximum and the cross-section of the jet may be nearly equal to that of the orifice itself.

The section at which this reduction in area is at a maximum is known as the "contracted vein" and experiment indicates that the velocity of the water follows Torricelli's law literally in this section. The section of the contracted vein generally lies at a distance from the orifice of five-tenths to eight-tenths of its least diameter.

Table 29, from Hughes & Safford's "Hydraulics," shows the approximate variation in coefficients of orifices for a circular orifice of diameter 0.033 ft. and for heads of from 1 to 10 ft.

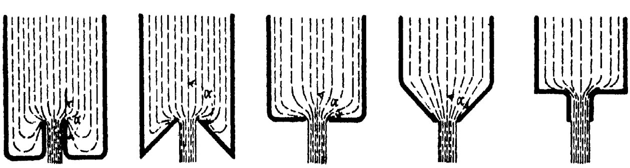


TABLE 29.—APPROXIMATE VARIATION IN COEFFICIENTS

	A		B		C	D					E
α	180°	157½°	135°	112½°	90°	67½°	45°	22½°	11¼°	5⅝°	0°
C	0.541	0.546	0.577	0.606	0.632	0.684	0.753	0.882	0.924	0.949	0.966

The standard orifice, as generally defined, is one in which the edge of the orifice which determines the jet is such that the jet upon leaving it

TABLE 30.—HAMILTON SMITH JR.'S COEFFICIENTS OF DISCHARGE FOR VERTICAL CIRCULAR ORIFICES

Applicable strictly to vertical circular orifices in a flat, rigid, thin wall, with full contraction of the jet; and for the dimensions and heads given. Direct interpolation should be made for intermediate values. "For heads of over 100 ft use $C = 0.592$"

Head from center of orifice	Diameter of the orifice in feet												
	0.02	0.03	0.04	0.05	0.07	0.10	0.12	0.15	0.20	0.40	0.60	0.80	1.0
Feet													
0.3	. . .	. . .	. . .	0.637	0.628	0.621	0.612	0.607	. . .	. . .	. . .	. . .	. . .
0.4	. . .	. . .	0.637	0.631	0.624	0.618	0.612	0.605	. . .	. . .	. . .	. . .	. . .
0.5	. . .	0.643	0.633	0.627	0.621	0.615	0.610	0.605	0.599	0.593	0.585	. . .	. . .
0.6	0.655	0.640	0.630	0.624	0.618	0.613	0.609	0.605	0.600	0.594	0.588	0.581	. . .
0.7	0.651	0.637	0.628	0.622	0.616	0.611	0.607	0.604	0.601	0.595	0.590	0.585	0.580
0.8	0.648	0.634	0.626	0.620	0.615	0.610	0.606	0.603	0.601	0.596	0.591	0.587	0.583
0.9	0.646	0.632	0.624	0.618	0.613	0.609	0.605	0.603	0.601	0.597	0.592	0.589	0.585
1.0	0.644	0.631	0.623	0.617	0.612	0.608	0.605	0.603	0.600	0.597	0.593	0.590	0.586
1.2	0.641	0.628	0.620	0.615	0.610	0.606	0.604	0.602	0.600	0.598	0.594	0.592	0.589
1.4	0.638	0.625	0.618	0.613	0.609	0.605	0.603	0.601	0.600	0.599	0.595	0.593	0.591
1.6	0.636	0.624	0.617	0.612	0.608	0.605	0.602	0.601	0.600	0.599	0.596	0.594	0.592
1.8	0.624	0.622	0.615	0.611	0.607	0.604	0.602	0.601	0.599	0.599	0.597	0.594	0.594
2.0	0.632	0.621	0.614	0.610	0.607	0.604	0.601	0.600	0.599	0.599	0.597	0.595	0.594
2.5	0.629	0.619	0.612	0.608	0.605	0.603	0.601	0.600	0.599	0.599	0.598	0.596	0.595
3.0	0.627	0.617	0.611	0.606	0.604	0.603	0.601	0.600	0.599	0.599	0.598	0.597	0.596
3.5	0.625	0.616	0.610	0.606	0.604	0.602	0.601	0.600	0.599	0.599	0.598	0.597	0.596
4.	0.623	0.614	0.609	0.605	0.603	0.602	0.600	0.599	0.599	0.598	0.597	0.597	0.596
5.	0.621	0.613	0.608	0.605	0.603	0.601	0.599	0.599	0.598	0.598	0.597	0.596	0.596
6.	0.618	0.611	0.607	0.604	0.602	0.600	0.599	0.599	0.598	0.598	0.597	0.596	0.596
7.	0.616	0.609	0.606	0.603	0.601	0.600	0.599	0.599	0.598	0.598	0.597	0.596	0.596
8.	0.614	0.608	0.605	0.603	0.601	0.600	0.599	0.598	0.598	0.597	0.596	0.596	0.596
9.	0.613	0.607	0.604	0.602	0.600	0.599	0.599	0.598	0.597	0.597	0.596	0.596	0.595
10.	0.611	0.606	0.603	0.601	0.599	0.598	0.598	0.597	0.597	0.597	0.596	0.596	0.595
20.	0.601	0.600	0.599	0.598	0.597	0.596	0.596	0.596	0.596	0.596	0.596	0.595	0.594
50 (?)	0.596	0.596	0.595	0.595	0.594	0.594	0.594	0.594	0.594	0.594	0.594	0.593	0.593
100 (?)	0.593	0.593	0.592	0.592	0.592	0.592	0.592	0.592	0.592	0.592	0.592	0.592	0.592

(Hughes and Safford's "Hydraulics," p. 144.)

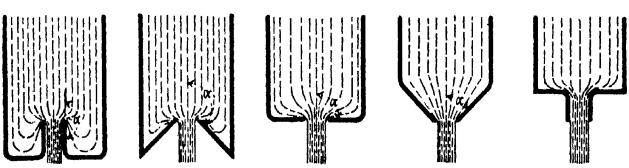
Discharge through Orifices.—In accordance with Torricelli's theorem, that the velocity of flow through the orifice is equal to the velocity acquired by a freely falling body in a space corresponding to the head over the orifice, the discharge through an orifice is as follows:

$Q = cav = ca\sqrt{2gh}$, in which
Q = quantity, in cubic feet per second
c = coefficient of discharge
a = net area of orifice, in square feet
v = velocity, in feet per second
h = head, in feet, from center of orifice to surface of water
g = acceleration of gravity = 32.16

The coefficient c is required by reason of the fact that the cross-section of the jet, at a point a short distance outside the orifice, has generally a somewhat smaller area than that of the orifice itself, the reduction in area depending upon the character of the orifice. When the edge of the orifice is sharp so that the water does not adhere to the orifice, the coefficient is at a minimum or the reduction in area is at a maximum. When, on the other hand, the orifice is shaped to a bell-mouth, the coefficient is at a maximum and the cross-section of the jet may be nearly equal to that of the orifice itself.

The section at which this reduction in area is at a maximum is known as the "contracted vein" and experiment indicates that the velocity of the water follows Torricelli's law literally in this section. The section of the contracted vein generally lies at a distance from the orifice of five-tenths to eight-tenths of its least diameter.

Table 29, from Hughes & Safford's "Hydraulics," shows the approximate variation in coefficients of orifices for a circular orifice of diameter 0.033 ft. and for heads of from 1 to 10 ft.



A. B. C. D. E.

TABLE 29.—APPROXIMATE VARIATION IN COEFFICIENTS

	A		B		C	D					E
α	180°	157½°	135°	112½°	90°	67½°	45°	22½°	11¼°	5⅝°	0°
C	0 541	0 546	0 577	0 606	0 632	0 684	0 753	0 882	0 924	0.949	0·966

The standard orifice, as generally defined, is one in which the edge of the orifice which determines the jet is such that the jet upon leaving it

does not again touch the wall of the orifice. Practically, this result is obtained by having the outside of the orifice bevelled and its throat cylindrical in shape with a cylinder length of between $\frac{1}{16}$ and $\frac{1}{8}$ in., depending upon the thickness of the plate.

Merriman defines it as signifying that: "The opening is so arranged that the water in flowing from it (the orifice) touches only a line as would be the case in a plate of no thickness. To secure this result, the inner edge of the opening has a square corner which alone is touched by the water. * * * * * The orifice in a thin plate is often used to express the condition that the water shall only touch the edges of the opening along the line. This arrangement may be regarded as a kind of standard apparatus for the measurement of water."

Hughes and Safford, "Hydraulics," p. 130, have, however, defined the standard orifice as follows:

"If an orifice in a thin wall is set far enough from the side of the vessel or channel to secure full contraction of the jet, is round or square, and has no dimensions greater than one foot (for which shapes and dimensions reliable coefficients are available), it is called a standard orifice."

Weirs.—One of the most accurate methods of measuring water is by the use of weirs, provided the conditions under which the coefficients of discharge of given types of weirs were determined are approximately reproduced in the gagings.

The most common types of weirs are the rectangular, the V-shaped and the trapezoidal weir.

The following data have been abstracted from Hughes and Safford's "Hydraulics," 1911, to which the reader is referred for a fuller discussion.

Procedure to be Followed in Weir Measurements.—(1) Constructing and setting up the weir and the gage for measuring the head; reproducing, if possible, the experimental conditions of the formula to be used.

(2) Measuring the length of the crest and determining its irregularities if any.

(3) Taking a profile of the crest if not sharp-edged.

(4) Determining by actual measurements the cross-sectional area of the channel of approach.

(5) Establishing by leveling the relative elevations of the crest of the weir, and the zero of the gage.

(6) When the desired regulation of flow is established, determining the head by hook gage or other observations at intervals as frequent as the conditions require.

(7) If possible, measure actual velocity in the channel of approach by a current meter or some other direct method, and

(8) Compute the discharge by the formula selected.

Three of these operations require especial consideration, viz., construction and setting, the measurement of the head, and the selection of the formula.

Construction and Setting of Weirs.—(1) A sharp-crested weir with complete crest contraction should be used.

(2) The crest should be level, and its ends vertical.

(3) The end contractions should be complete, or, if suppressed, entirely suppressed.

(4) The upstream face should be vertical; the downstream so designed that the nappe has free overfall.

(5) Free access for air under the nappe should be made certain.

(6) The weir should be set at right angles to the direction of flow.

(7) The channel of approach should be straight for at least 25 ft. above the weir, of practically uniform cross-section, and of slight slope (preferably none).

(8) Screens of coarse wire or baffles of wood should be set in the channel, if necessary, to equalize the velocities in different parts of the channel, but not nearer the crest than 25 ft.

(9) The channel of approach should have a large cross-sectional area in order to keep the velocity of approach low.

Measurement of Head.—The head above the crest of the weir should be measured, preferably, by a hook gage with vernier scale upon it, reading to thousandths of a foot.

For approximate results, the gagings may be made from a peg driven into the bed of the stream at a distance of several feet above and to one side of the weir. But for careful or precise measurements the gagings must be made in a still box, the location of which should meet the following essential conditions:

(1) The cross-sectional area of the communicating opening or pipe must be sufficient to allow free communication with the channel even when throttled.

(2) The channel end of this opening must be set into and exactly flush with the flat walls of the channel, or into a flat surface laid parallel to the direction of flow, and the pipe itself must be normal to the direction of flow.

(3) The channel end of this opening must be located far enough upstream to avoid the slope of the surface curve, and not far enough to increase the observed head by the natural slope of the stream.

The area of increased pressure, which forms above the bottom, beginning at the upstream face of the weir and extending upstream, perhaps about to the beginning of the surface curve,[1] once thought to be a location at which the observed head would include the velocity head, has been proved to be a poor location for the opening.

Avoid perforated pipes, no matter where the holes are bored, laid transversely or longitudinally in the stream at different depths; avoid so-called piezometers of any form which project in any direction into the stream. After the Lowell hydraulic experiments were made, Francis sometimes used pipes with holes bored in a vertical plane in order to secure an average pressure across the stream, in recognition of the fact that the surface is not

[1] See Fteley and Stearns, Trans. Am. Soc. C. E. Vol 12, p 42, Plate IV.

transversely level. Since his time, this has been shown to be a vicious practice, which may introduce more errors than it was designed to obviate.

The essential conditions of location of a still box will in general be met if its opening is set well upstream from the beginning of the surface curve, and at or a few inches below the crest level.

If Francis's, Fteley and Stearns', Bazin's, or any particular experimenter's formula is to be used, his location should be duplicated (p. 200, Hughes and Safford's Hydraulics).

Measurements of Head in Francis's Experiments.—The head was observed by two hook gages, one on each side of the channel, set in still boxes which were 18 in. long by 11 in. wide. Communication with the channel was made for the contracted weir measurements by a 1-in. diameter hole in the bottom of each box, located 6 ft. upstream from the weir and 4 in. lower than the level of the crest. For the suppressed weir, communication was established by pipes *B*, opening into the sides of the channel 1 ft. lower than the level of the crest, or by the single opening for the pipes 4 and 5 which were set in the board *C*. * * * * * All three openings used were therefore 6 ft. upstream from the weir. To prevent rapid oscillations, the openings were throttled by a perforated plug set on the inside of the still boxes (p. 204, loc. cit.).

Measurements of Head in Fteley and Stearns' Experiments.—* * * * * the head was measured by hook gages set in still boxes which were connected with the channel by pipes. Although the actual form of piezometer openings varied, the essential condition that the opening be at or below the crest in and normal to a flat surface parallel to the direction of flow, was in all cases maintained. The location of each opening is stated in the table" (p. 208, loc. cit.).

The General Weir Formula may be expressed by the equation $Q = CLH^{\frac{3}{2}}$. To this form all the equations in use may be reduced, but it is better practice, in view of the several methods of correcting for the velocity of approach followed by the various experimenters, to use their form of equation.

The Francis Weir formula.

Let Q = discharge in cubic feet per second,
L = length of crest of weir in feet,
N = number of end contractions,
H = the observed head corrected to include the effect of the velocity of approach,
h = the observed head upon the crest of weir, being the difference in elevation in feet between the top of the crest and the surface of the water in the channel, at a point upstream, which should, if possible, be taken just beyond the beginning of the surface curve,
h_v = the head due to the mean velocity of approach
$= V_A^2/2g$ in feet per second.

Table 32.—Weir Discharges and Velocities due to Heads from 0.00 to 2.99 ft.

(Safford)

Quantity of water in cubic feet per second, discharged over a weir 1 ft. long, the weir to have complete contraction on its crest, and to have no end contractions. $Q = 3.31\ L\ H^{\frac{3}{2}} + 0.007\ L$ for depths up to 0.5′ and $Q = 3.33\ L\ H^{\frac{3}{2}}$ for depths above 0.5 ft. *Vel.* due to head computed by formula $Vel. = \sqrt{2gh}$

H feet	Quan. c. f., p. s.	Velo. ft. p. s.	H feet	Quan. c. f., p. s.	Velo. ft. p. s.	H feet	Quan. c. f., p. s.	Velo. ft. p. s.	H feet	Quan. c. f., p. s.	Velo. ft. p. s.	H feet	Quan. c. f., p. s.	Velo. ft. p. s.
0.00	. . .	. . .	**0.60**	1 55	6 21	**1 20**	4.38	8 79	**1.80**	8 04	10.76	**2.40**	12.38	12 42
.01	0.01	0 80	**.61**	.59	6 26	**.21**	.43	8 82	**.81**	.11	10 79	**.41**	.46	12 45
.02	.02	1 13	**.62**	.63	6 32	**.22**	.49	8 86	**.82**	.18	10 82	**.42**	.54	12.48
.03	.02	1 39	**.63**	.66	6 37	**.23**	.54	8 89	**.83**	.24	10 85	**.43**	.61	12.50
.04	.03	1 60	**.64**	.70	6.42	**.24**	.60	8.93	**.84**	.31	10 88	**.44**	.69	12.53
.05	.04	1 79	**.65**	.74	6 47	**.25**	.65	8 97	**.85**	.38	10 91	**.45**	.77	12 55
.06	.06	1 96	**.66**	.79	6 52	**.26**	.71	9 00	**.86**	.45	10 94	**.46**	.85	12.58
.07	.07	2 12	**.67**	.83	6 56	**.27**	.77	9 04	**.87**	.51	10 97	**.47**	93	12 60
.08	.08	2 27	**.68**	.87	6 61	**.28**	.82	9 07	**.88**	.58	11 00	**48**	13 00	12 63
.09	.10	2.41	**.69**	.91	6.66	**.29**	.88	9.11	**.89**	.65	11.03	**.49**	.08	12.66
0.10	0.11	2 54	**0 70**	1 95	6.71	**1.30**	4.94	9 14	**1.90**	8.72	11 05	**2 50**	13.16	12 68
.11	.13	2.66	**.71**	.99	6 76	**.31**	.99	9 18	**.91**	.79	11 08	**.51**	.24	12.71
.12	.14	2 78	**.72**	2.03	6.81	**.32**	5.05	9 21	**.92**	.86	11 11	**.52**	.32	12 73
.13	.16	2 89	**.73**	.08	6 85	**.33**	.11	9 25	**.93**	93	11 14	**.53**	.40	12 76
.14	.18	3 00	**.74**	.12	6 90	**.34**	.16	9 28	**.94**	9 00	11 17	**.54**	.48	12.78
.15	.20	3 11	**.75**	.16	6 95	**.35**	.22	9 32	**.95**	.07	11 20	**.55**	.56	12.81
.16	.22	3 21	**.76**	.21	6 99	**.36**	.28	9 35	**.96**	.14	11 23	**.56**	.64	12.83
.17	.24	3 31	**.77**	.25	7 04	**.37**	.34	9 39	**.97**	.21	11 26	**.57**	.72	12 86
.18	.26	3.40	**.78**	.29	7 08	**.38**	40	9 42	**.98**	.28	11 29	**58**	80	12 88
.19	.28	3.50	**.79**	.34	7.13	**.39**	.46	9.46	**.99**	.35	11 31	**.59**	.88	12.91
0.20	0.30	3 59	**0.80**	2.38	7 17	**1.40**	5.52	9 49	**2 00**	9 42	11 34	**2 60**	13 96	12 93
.21	.33	3 68	**.81**	.43	7.22	**.41**	.57	9 52	**.01**	.49	11 37	**.61**	14 01	12 96
.22	.35	3 76	**.82**	.47	7 26	**.42**	.63	9 56	**.02**	.56	11 40	**.62**	.12	12 98
.23	.37	3 85	**.83**	.52	7 31	**.43**	.69	9 59	**.03**	.63	11.43	**.63**	.20	13.01
.24	.40	3.93	**.84**	.56	7 35	**.44**	.75	9 62	**.04**	.70	11 46	**.64**	.28	13 03
.25	.42	4 01	**.85**	.61	7 39	**.45**	.81	9.66	**.05**	.77	11 48	**.65**	.36	13 06
.26	.45	4 09	**.86**	.66	7 44	**.46**	.87	9 69	**.06**	.85	11 51	**66**	.45	13 08
.27	.47	4 17	**.87**	.70	7 48	**.47**	93	9 72	**.07**	92	11 54	**.67**	.53	13 11
.28	.50	4 21	**.88**	.75	7 52	**.48**	6.00	9 76	**.08**	.99	11 57	**68**	.61	13 13
.29	.52	4.32	**.89**	.80	7.57	**.49**	.06	9.79	**.09**	10.06	11.59	**69**	.69	13.15
0.30	0 55	4.39	**0.90**	2 84	7 61	**1.50**	6 12	9 82	**2.10**	10.13	11 62	**2.70**	14 77	13 18
.31	.58	4 47	**.91**	.89	7 65	**.51**	.18	9 86	**.11**	.21	11 65	**.71**	.86	13 20
.32	.61	4 54	**.92**	.94	7 69	**.52**	.24	9.89	**.12**	.28	11 68	**.72**	.91	13 23
.33	63	4.61	**.93**	.99	7.73	**.53**	.30	9.92	**.13**	.35	11 70	**.73**	15.02	13 25
.34	.66	4 68	**.94**	3.03	7.78	**.54**	.36	9.95	**.14**	.42	11.73	**.74**	.10	13.28
.35	.69	4.74	**.95**	.08	7.82	**.55**	.43	9.98	**.15**	.50	11 76	**.75**	.19	13.30
.36	.72	4 81	**.96**	.13	7.86	**.56**	.49	10.02	**.16**	.57	11.79	**.76**	.27	13 32
37	.75	4 88	**.97**	.18	7.90	**.57**	.55	10.05	**.17**	.64	11 81	**.77**	.35	13 35
.38	.78	4 91	**.98**	.23	7 94	**.58**	.61	10 08	**.18**	.72	11 84	**.78**	.43	13.37
.39	.81	5.01	**.99**	.28	7.98	**.59**	.68	10.11	**.19**	.79	11.87	**.79**	.52	13.40
0.40	0 84	5.07	**1.00**	3.33	8.02	**1.60**	6.74	10.14	**2.20**	10.87	11 90	**2 80**	15.60	13.42
.41	.88	5.14	**.01**	.38	8 06	**.61**	.80	10 18	**.21**	.94	11 92	**.81**	.69	13 44
.42	.91	5 20	**.02**	.43	8 10	**.62**	.87	10.21	**.22**	11.01	11.95	**.82**	.77	13.47
.43	.94	5 26	**.03**	.48	8.14	**.63**	.93	10.24	**.23**	.09	11.98	**.83**	.85	13.49
.44	.97	5.32	**.04**	.53	8.18	**.64**	.99	10 27	**.24**	.16	12 00	**.84**	.94	13.52
.45	1.01	5 38	**.05**	.58	8.22	**.65**	7.06	10 30	**.25**	.24	12 03	**.85**	16.02	13 54
.46	.04	5 44	**.06**	.63	8.26	**.66**	.12	10 33	**.26**	.31	12 06	**.86**	.11	13.56
.47	.07	5 50	**.07**	.69	8.30	**.67**	.19	10 36	**.27**	.39	12.08	**.87**	.19	13 59
.48	.11	5.56	**.08**	.74	8.33	**.68**	.25	10.40	**.28**	.46	12.11	**.88**	.27	13 61
.49	.14	5.61	**.09**	.79	8.37	**.69**	.32	10.43	**.29**	.54	12.14	**.89**	.36	13.63
0.50	1.18	5.67	**1.10**	3.84	8.41	**1.70**	7.38	10.46	**2.30**	11.61	12.16	**2.90**	16.44	13.66
.51	.21	5.73	**.11**	.89	8.45	**.71**	.45	10 49	**.31**	.69	12.19	**.91**	.53	13.68
.52	.25	5 78	**.12**	.95	8.49	**.72**	.51	10 52	**.32**	.77	12.22	**.92**	.62	13.70
.53	.28	5.84	**.13**	4.00	8.53	**.73**	.58	10 55	**.33**	.84	12.24	**.93**	.70	13.73
.54	.32	5.89	**.14**	.05	8.56	**.74**	.64	10.58	**.34**	.92	12.27	**.94**	.79	13.75
.55	.36	5 95	**.15**	.11	8.60	**.75**	.71	10 61	**.35**	12.00	12.29	**.95**	.87	13.78
.56	.39	6.00	**.16**	.16	8.64	**.76**	.77	10 64	**.36**	.07	12.32	**.96**	.96	13.80
.57	.43	6.06	**.17**	.21	8.68	**.77**	.84	10.67	**.37**	.15	12 35	**.97**	17.04	13.82
.58	.47	6.11	**.18**	.27	8 71	**.78**	.91	10 70	**.38**	.23	12.37	**.98**	.13	13 84
.59	.51	6.16	**.19**	.32	8 75	**.79**	.97	10.73	**.39**	.30	12.40	**.99**	.22	13.87

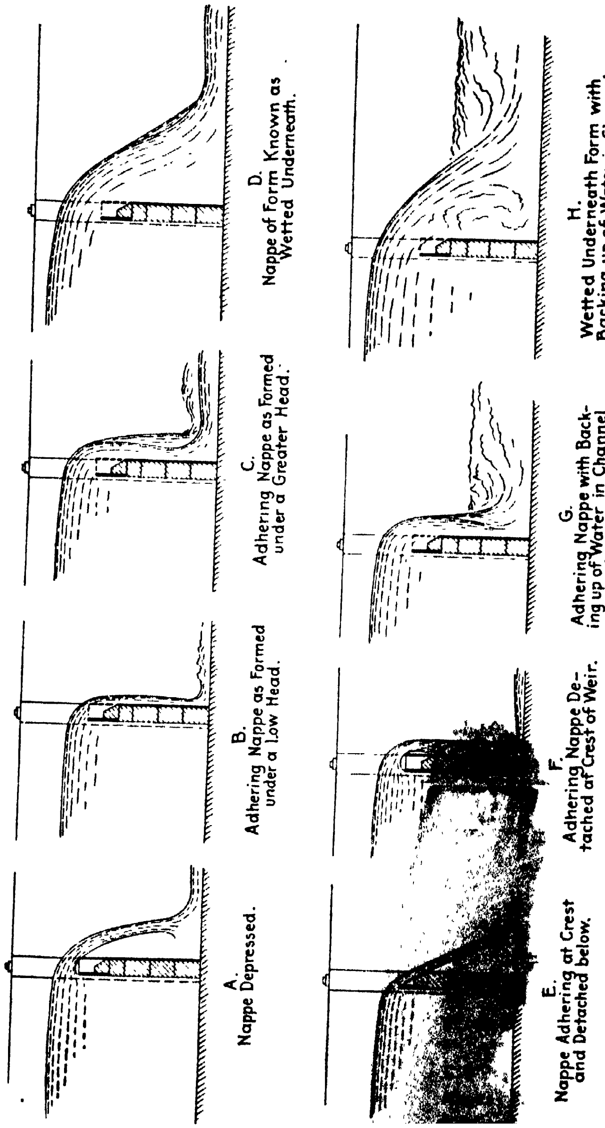


FIG. 42.—Forms of Nappe occurring if air is cut off from beneath the sheet.
(Hughes and Safford, from Experiments by Bazin.)
As compared with free overfall, the discharge may be increased with A as much as 6 per cent.; with D, 15 per cent: with G, 25 per cent.

For contracted weirs, neglecting velocity of approach:

$$Q = 3.33\ (L - 0.1\ Nh)h^{\frac{3}{2}}$$

Note.—The use of h instead of H in the factor $(L - 0.1NH)$ used in correcting for end contractions is as precise as ordinary practice warrants.

For contracted weirs, head corrected for velocity of approach:

$$Q = 3.33\ (L - 0.1NH)\ [(h + h_v)^{\frac{3}{2}} - h_v^{\frac{3}{2}}]$$

For suppressed weirs, neglecting velocity of approach:

$$Q = 3.33Lh^{\frac{3}{2}}$$

For suppressed weirs, head corrected for velocity of approach:

$$Q = 3.33L[(h + h_v)^{\frac{3}{2}} - h_v^{\frac{3}{2}}]$$ (p. 205, loc. cit.).

The Fteley and Stearns formula

$$Q = 3.31\ LH^{\frac{3}{2}} + 0.007L$$
$$H = (h + 1.50\ h_v) \text{ for suppressed weirs}$$
$$H = (h + 2.05\ h_v) \text{ for contracted weirs}$$

For contracted weir make $L = (L - 0.1NH)$

The *H. Smith, Jr., formula*

$$Q = (c_s \text{ or } c_c)\,\tfrac{2}{3}L(2g)^{\frac{1}{2}}H^{\frac{3}{2}}$$
$$H = (h + 1\tfrac{1}{3}h_v) \text{ for suppressed weirs}$$
$$H = (h + 1.4h_v) \text{ for contracted weirs}$$

The *Bazin formula*

$$Q = mLh(2gh)^{\frac{1}{2}} \text{ (for suppressed weirs only)}$$

m = coefficient including effects of crest contraction and of velocity of approach. (p. 202, loc. cit.)

The Francis formulas are strictly applicable only to vertical sharp-crested rectangular weirs with complete contractions and with free overfall and

When the head (H) is not greater than one-third the length (L);

When the head is not less than 0.5 ft. nor more then 2 ft;

When the velocity of approach is 1 ft. per second or less;

When the height of the weir is at least three times the head.

In all probability the formulas are usable with higher heads than 2 ft., but not much lower than 0.5 ft., as shown by Fteley and Stearns' experiment (p. 207, loc. cit.).

Choice of Formulas.—"When Francis's weir settings can be duplicated or the velocity of approach is very low, 1 ft. per second or less, there is general willingness on the part of both engineers and laymen to accept this formula for heads for from 0.5 to 2 ft., and the same is true of the Fteley and Stearns formula for heads of 0.07 to 0.5 ft. For higher heads the Cornell experiments, which are the only guides, indicate that the Francis formula may be used with reasonable accuracy up to heads of 5 ft.

Bazin's formula is the best where his conditions can be reproduced and

if the velocity of approach is high and the height of weir low, his formula is the only one sufficiently flexible. For this reason it is the most useful.

Smith's coefficients are the result of the most thorough study, but are based upon experimental data of somewhat unequal accuracy. They do, however, furnish means for satisfactory interpolation to suit cases not covered precisely by the data which he used.

If possible, contracted weirs should be avoided, but are often necessary to insure atmospheric pressure underneath the nappe; if end contractions are unavoidable, the Francis formula should be used.

For rough measurements there has never appeared to be any good reason for departing from the Francis formula, which has the advantage of long usage and consequent familiarity, especially in legal cases, although it has often been used far beyond the limits laid by Mr. Francis himself. It should be borne in mind, however, that his formula applies only to sharp-crested weirs" (page 223, loc. cit.).

Triangular Weirs.—The theoretic discharge of the triangular weir is given by the equation,

$$Q = \frac{4}{15} L (2g)^{\frac{1}{2}} h^{\frac{3}{2}}$$

in which Q = discharge in cubic feet per second
L = length of crest at level of h or water surface
h = head over angle of the weir notch in feet

Prof. James Thomson deduced experimentally a value of $C = 0.617$ for

TABLE 33.—DISCHARGE OF RIGHT-ANGLE TRIANGULAR WEIR.

$$Q = 2.54\ H^{\frac{5}{2}} \text{ cu. ft. per second.}$$

Head in feet	0.0	0.1	0.2	0.3	0.4	0.5	0.6
0.00		0.0080	0.0454	0.1252	0.2570	0.4490	0.7083
0.005	..	0.0097	0.0483	0.1305	0.2651	0.4603	0.7231
0.01	.	0.0102	0.0513	0.1359	0.2734	0.4718	0.7382
0.015	.	0.0114	0.0544	0.1415	0.2818	0.4834	0.7534
0.020	0.00014	0.0127	0.0577	0.1471	0.2904	0.4953	0.7688
0.025	0.00025	0.0140	0.0610	0.1529	0.2991	0.5073	0.7844
0.030	0.00010	0.0155	0.0644	0.1589	0.3080	0.5194	0.8002
0.035	0.00058	0.0170	0.0680	0.1650	0.3170	0.5318	0.8162
0.040	0.00081	0.0186	0.0717	0.1712	0.3262	0.5443	0.8323
0.045	0.00109	0.0203	0.0755	0.1776	0.3355	0.5570	0.8487
0.050	0.00142	0.0221	0.0794	0.1841	0.3450	0.5698	0.8652
0.055	0.00180	0.0240	0.0834	0.1907	0.3547	0.5829	0.8820
0.060	0.00219	0.0260	0.0876	0.1975	0.3645	0.5961	0.8989
0.065	0.00274	0.0281	0.0918	0.2044	0.3745	0.6095	0.9160
0.070	0.00329	0.0303	0.0962	0.2115	0.3847	0.6231	0.9333
0.075	0.00391	0.0325	0.1008	0.2188	0.3950	0.6368	0.9508
0.080	0.00460	0.0349	0.1054	0.2261	0.4055	0.6508	0.9685
0.085	0.00535	0.0374	0.1102	0.2336	0.4161	0.6649	0.9864
0.090	0.00617	0.0400	0.1150	0.2412	0.4269	0.6791	1.0045
0.095	0.00707	0.0427	0.1201	0.2491	0.4379	0.6936	1.0228

heads of 0.2 to 0.8 ft., in which case the formula would reduce to the form

$$Q = 1.32\ Lh^{\frac{3}{2}}$$

and for right-angled notches in which $L = 2\ h$; $Q = 2.64\ h^{\frac{5}{2}}$.

Experiments made at the Massachusetts Institute of Technology, under the direction of Professor Dwight Porter, gave for the right-angled notched weir,

$$Q = 2.54\ h^{\frac{5}{2}}$$

Trapezoidal Weirs.—The trapezoidal weir differs from the rectangular type in that the sides, instead of being vertical, are built upon a slope. Usually the slope is built with a batter of 1 in 4 for the reason that at this angle the slope is just about sufficient to offset the effect of end contractions. When this is done the weir is known as the "Cippoletti Weir." The general equation of the trapezoidal weir is as follows:

$$Q = \frac{2}{3}\ (2g)^{\frac{1}{2}}\ Lh^{\frac{3}{2}} + \frac{4}{15}\ 2z(2g)^{\frac{1}{2}}\ h^{\frac{5}{2}}$$

in which Q = quantity in cubic feet per second

L = length of the weir at the bottom of the notch in feet

h = the head of water over the notch in feet

z = the batter of the side or the ratio of the vertical projection to the horizontal projection of the side

g = the gravity = 32.16

For the Cippoletti, in which $z = \frac{1}{4}$, the formula reduces to

$$Q = 3.367\ Lh^{\frac{3}{2}}$$

Irregular Weirs.—For the determination of the discharge over broad crested weirs and dams having different types of crests, reference may be had to an admirable digest of "Weir Experiments, Coefficients, and Formulas" by Robert E. Horton, published as Water Supply and Irrigation Paper No. 200, of the U. S. Geological Survey, 1907, and to standard works upon Hydraulics.

Venturi Meter.—The principle of this apparatus, based upon Bernoulli's theorem, was discovered about 1791 by the Italian engineer, J. B. Venturi, who stated that when fluids or gases discharged through an expanding nozzle a sucking action was exercised in the small diameter, diminishing as the diameter increases. This principle was first practically applied by Clemens Herschel in 1887 in the so-called Venturi meter. The meter tube, which is the portion of the apparatus to which Venturi's discovery applies, is inserted in a line of pipe and consists of three parts, the inlet cone, in which the diameter of the pipe is gradually reduced, the throat or constricted section, and the outlet cone, in which

the diameter increases gradually to that of the pipe in which the meter is inserted. The throat is lined with bronze; its diameter, in standard meter tubes, is from one-third to one-half of the diameter of the pipe; and its length but a few inches, sufficient to allow a suitable pressure chamber or piezometer ring to be inserted in the pipe at this point. The upper or inlet cone has a length of approximately one-fourth that of the lower cone. A piezometer ring is inserted at the upper or large end of the inlet cone, and the determination of the quantity of water flowing is based upon the difference in pressures observed or indicated at this point and at the throat of the meter. The general form of the meter is shown in Fig. 43.

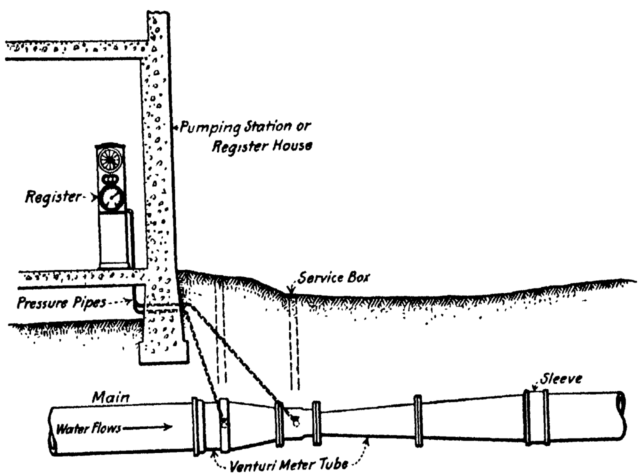


FIG. 43.—Arrangement of venturi meter on pressure pipe.

The derivation of a formula from which the discharge of the Venturi meter tube is computed may be found in Hughes and Safford's "Hydraulics," p. 116. As written by Herschel the form of this expression is

$$Q = \frac{a_1 a_2}{\sqrt{a_1^2 - a_2^2}} \sqrt{2g(h_1 - h_2)}$$

$$= \frac{a_1 a_2}{\sqrt{a_1^2 - a_2^2}} \sqrt{2gH}$$

in which $a_1 a_2$ are the areas in square feet at the upstream end and at the throat of the meter, respectively, $h_1 h_2$ the pressure heads at the corresponding points,

$$H = h_1 - h_2$$

Under actual operating conditions, and for standard meter tubes, including allowance for friction, this formula reduces to the form

$$Q = (1.00 \pm 0.02) a_2 \sqrt{2gH}$$

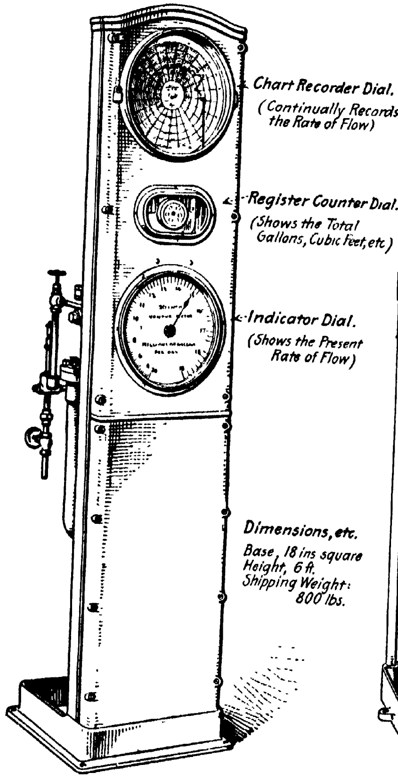


FIG. 44.—Type M register-indicator recorder.

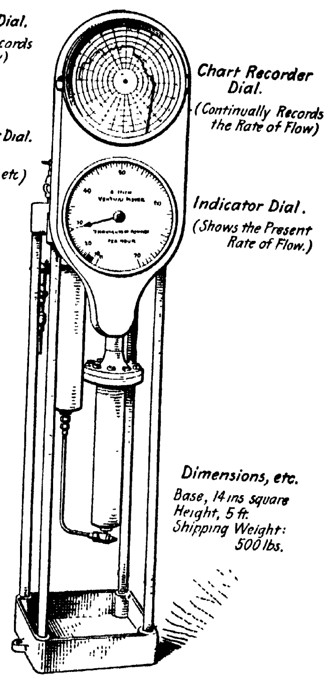


FIG. 45.—Type M indicator-recorder.

The coefficient written (1.00 ± 0.02) is made up of two parts, or $C = C_1 C_2$.

$$C_1 = \frac{a_1}{\sqrt{a_1^2 - a_2^2}}$$

$$C_2 = \text{coefficient of friction.}$$

For standard meter tubes in which the diameter of the throat is between one-third and one-half that of the pipe, the values of C_1 range

between 1.0062 and 1.0328, while the friction coefficient C_2 varies from 0.97 to 0.99. Thus the range of values of C is from 0.98 to 1.02, and

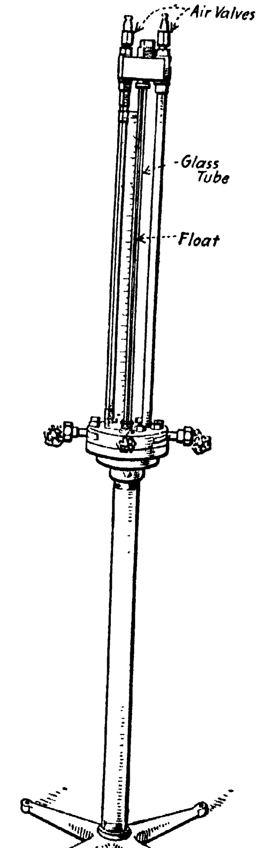


Fig. 46.—Manometer for venturi meter.

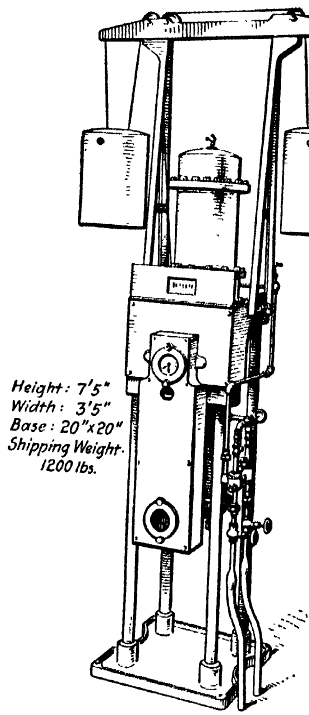


Fig. 47.—Type D register and chart recorder.

accordingly C has been written above as (1.00 ± 0.02). Hazen[1] thinks $C = 0.99$ the best value for practical use. J. W. Ledoux[2] gives results

[1] *Eng. News*, July 31, 1913, p. 199.
[2] *Eng. News*, July 31, 1913, p. 201.

of experiments made by him on a 4-in. meter tube with 2-in. throat, which show a coefficient less than 0.98, approximating 0.975 for ordinary velocities and falling as low as 0.915 for very low velocities, about 0.67 ft. per second, through the throat of the tube.

The Venturi meter affords one of the most accurate methods of measuring water, the registration being within 2 per cent. of the actual flow of water at ordinary velocities.

The Venturi meters are made by Builders Iron Foundry of Providence, R. I., under Herschel's and their own patents. The standard sizes and approximate cost of the meters, with cast-iron meter tubes, are shown in Table 34, the list prices being sufficiently close to actual prices for preliminary estimate. For sizes of 48 in. and larger it is often possible to construct a meter tube at less cost by using in part some other material than cast iron, such as concrete or steel plates. In the table $50 is included as the cost of the oil seal in each case. The prices given also apply for float-operated instruments when the cost of the float pipes is included.

TABLE 34.—VENTURI METER TUBE PRICES (F.O.B. PROVIDENCE) FOR PRESSURES UP TO 125 LB. PER SQUARE INCH, INCLUDING CLEANING DEVICE AND OIL SEALS

Diameter, inches	Price	Diameter, inches	Price
6	$150 00	32	$ 895.00
8	180.00	34	995.00
10	205.00	36	1100.00
12	245.00	38	1205 00
14	285.00	40	1320.00
16	360.00	42	1440.00
18	420.00	44	1570.00
20	480.00	46	1700.00
22	550.00	48	1835.00
24	620.00	54	2290.00
26	645.00	60	2790.00
28	720.00	66	3345.00
30	810.00	72	3950.00

Type M Indicator-recorder	$290.00
Type M Register-indicator-recorder	450.00
Special planimeter	30 00
Manometer	65.00

A similar meter, using the Venturi tube but having a different recording mechanism, has recently (1913) been put upon the market by the Simplex Valve & Meter Co. The principle of its recording device was described by J. W. Ledoux in Transactions Am. Soc. C. E., Vol. LXXVI, p. 1048.

The minimum measuring capacity of the Venturi meter, in U. S. gallons per day, is approximately equal to the square of the throat diameter in inches, followed by four ciphers, thus: 4-in. throat diameter, $4 \times 4 = 16$; minimum measuring capacity = 160,000 gal. per day; 8-in. throat diameter, 640,000 gal. per day.

The maximum measuring capacity of the meter is approximately thirteen times the minimum capacity.

The actual loss in pressure corresponding to the maximum discharge of these meters as built is approximately 1 lb. per square inch, so that under operating conditions the loss in pressure of the water, due to friction in the meter, is generally not over 3/4 lb. per square inch. Greater quantities than the listed maximums may be discharged through the meter tube with a loss of head proportional to the squares of the quantities. Thus, a 24-in. meter tube (recorded in Table 35 as Throat Diameter 10) has a friction loss of 1 lb. per square inch when discharging at its maximum measuring capacity of 13 million gallons per day. At half this rate the friction loss is 1/4 lb. per square inch. Similar calculations can be made for any other rates of flow.

TABLE 35.—VENTURI METER DATA FOR DESIGNERS

Inlet and outlet diameter, inches	Throat diameter	Length	Measuring capacity: Gallons per 24 hours, Minimum	Measuring capacity: Gallons per 24 hours, Maximum	Measuring capacity: Gallons per minute, Minimum	Measuring capacity: Gallons per minute, Maximum	Approximate weight, pounds
2	⅝	1 ft. 11⅝ in.	4,000	51,000	3	35	50
	¾	1 ft. 10¼ in.	6,000	73,000	4	55	
	1	1 ft. 7 in.	10,000	130,000	7	90	
2½	½ A	2 ft. 4⅜ in.	7,000	100,000	5	70	85
	½ B	2 ft. 3 in.	10,000	130,000	7	90	
	½ C	1 ft. 11¾ in.	16,000	203,000	11	140	
3	1	2 ft. 11 in.	10,000	130,000	7	90	110
	1¼	2 ft. 7¾ in.	16,000	203,000	11	140	
	1½	2 ft. 4½ in.	23,000	293,000	16	205	
4	1¼	4 ft. 3⅞ in.	16,000	203,000	11	140	160
	1⅝	3 ft. 10¼ in.	26,000	343,000	18	240	
	2	3 ft. 6 in.	40,000	520,000	28	360	
5	1⅝	5 ft. 1⅜ in.	26,000	343,000	18	240	275
	2	4 ft. 8½ in.	40,000	520,000	28	360	
	2½	4 ft. 2 in.	63,000	813,000	44	565	
6	2	5 ft. 11 in.	40,000	520,000	28	360	450
	2½	5 ft. 4½ in.	63,000	813,000	44	565	
	3	4 ft. 10 in.	90,000	1,170,000	63	810	
8	2¾	7 ft. 6¼ in.	76,000	983,000	53	680	700
	3¼	6 ft. 11¾ in.	106,000	1,373,000	74	950	
	4	6 ft. 2 in.	160,000	2,080,000	110	1,440	

TABLE 35.—VENTURI METER DATA FOR DESIGNERS. *(Continued)*

Inlet and outlet diameter inches	Throat diameter	Length	Measuring capacity: Gallons per 24 hours, Minimum	Measuring capacity: Gallons per 24 hours, Maximum	Measuring capacity: Gallons per minute, Minimum	Measuring capacity: Gallons per minute, Maximum	Approximate weight, pounds
10	3¼	9 ft 4¼ in	106,000	1,373,000	74	950	1,100
	4	8 ft. 7 in	160,000	2,080,000	110	1,440	
	5	7 ft. 6 in	250,000	3,250,000	175	2,260	
12	4	11 ft. 0 in	160,000	2,080,000	110	1,440	1 550
	5	9 ft. 11 in	250,000	3,250,000	175	2,260	
	6	8 ft 10 in	360,000	4,680,000	250	3,250	
14	4½	12 ft 10½ in	203,000	2,633,000	140	1,830	2,200
	5¾	11 ft. 6¾ in	321,000	4,298,000	230	2,980	
	7	10 ft 2 in.	490,000	6,370,000	340	4,420	
16	5¼	14 ft 5¼ in	276,000	3,583,000	190	2,490	3,000
	6½	13 ft 1½ in	423,000	5,493,000	295	3,810	
	8	11 ft 6 in	640,000	8,320,000	445	5,780	
18	6	16 ft 1 in	360,000	4,680,000	250	3,250	3,700
	7½	14 ft 5½ in	563,000	7,313,000	390	5,080	
	9	12 ft 10 in	810,000	10,530,000	560	7,310	
20	6½	17 ft 11½ in	423,000	5,493,000	295	3,810	4,750
	8	16 ft 4 in	640,000	8,320,000	445	5,780	
	10	14 ft 2 in	1,000,000	13,000,000	695	9,020	
22	7	19 ft 10 in	490,000	6,370,000	340	4,420	5,700
	9	17 ft 8 in	810,000	10,530,000	560	7,310	
	11	15 ft. 6 in	1,210,000	15,730,000	840	10,900	
24	8	21 ft 2 in.	640,000	8,320,000	445	5,780	6,800
	10	19 ft. 0 in	1,000,000	13,000,000	695	9,020	
	12	16 ft 10 in	1,440,000	18,720,000	1,000	13,000	
26	8½	23 ft. 0½ in	723,000	9,393,000	500	6,520	8,300
	11	20 ft 4 in	1,210,000	15,730,000	840	10,900	
	13	18 ft 2 in	1,690,000	21,970,000	1,170	15,300	
28	9	24 ft. 11 in	810,000	10,530,000	560	7,310	9,600
	11½	22 ft. 2½ in.	1,323,000	17,193,000	920	11,900	
	14	19 ft. 6 in	1,900,000	25,480,000	1,360	17,700	
30	10	26 ft. 3 in.	1,000,000	13,000,000	695	9,020	11,000
	13	23 ft 0 in	1,690,000	21,970,000	1,170	15,300	
	15	20 ft. 10 in	2,250,000	29,250,000	1,560	20,300	
32	10½	28 ft 1½ in	1,103,000	14,333,000	765	9,950	12,700
	13	25 ft. 5 in.	1,690,000	21,970,000	1,170	15,300	
	16	22 ft 2 in	2,560,000	33,280,000	1,780	23,100	
34	11	30 ft. 0 in	1,210,000	15,730,000	840	10,900	14,300
	14	26 ft. 9 in	1,960,000	25,480,000	1,360	17,700	
	17	23 ft 6 in	2,890,000	37,570,000	2,010	26,100	

The minimum measuring capacity of the Venturi meter, in U. S. gallons per day, is approximately equal to the square of the throat diameter in inches, followed by four ciphers, thus: 4-in. throat diameter, $4 \times 4 = 16$; minimum measuring capacity = 160,000 gal. per day; 8-in. throat diameter, 640,000 gal. per day.

The maximum measuring capacity of the meter is approximately thirteen times the minimum capacity.

The actual loss in pressure corresponding to the maximum discharge of these meters as built is approximately 1 lb. per square inch, so that under operating conditions the loss in pressure of the water, due to friction in the meter, is generally not over 3/4 lb. per square inch. Greater quantities than the listed maximums may be discharged through the meter tube with a loss of head proportional to the squares of the quantities. Thus, a 24-in. meter tube (recorded in Table 35 as Throat Diameter 10) has a friction loss of 1 lb. per square inch when discharging at its maximum measuring capacity of 13 million gallons per day. At half this rate the friction loss is 1/4 lb. per square inch. Similar calculations can be made for any other rates of flow.

Table 35.—Venturi Meter Data for Designers

Inlet and outlet diameter, inches	Throat diameter	Length	Measuring capacity: Gallons per 24 hours, Minimum	Measuring capacity: Gallons per 24 hours, Maximum	Measuring capacity: Gallons per minute, Minimum	Measuring capacity: Gallons per minute, Maximum	Approximate weight, pounds
2	⅝	1 ft. 11⅜ in	4,000	51,000	3	35	50
	¾	1 ft. 10¼ in	6,000	73,000	4	55	
	1	1 ft 7 in.	10,000	130,000	7	90	
2½	½ A	2 ft. 4⅜ in.	7,000	100,000	5	70	85
	½ B	2 ft. 3 in.	10,000	130,000	7	90	
	½ C	1 ft. 11¾ in	16,000	203,000	11	140	
3	1	2 ft 11 in.	10,000	130,000	7	90	110
	1¼	2 ft. 7¾ in	16,000	203,000	11	140	
	1½	2 ft. 4½ in.	23,000	293,000	16	205	
4	1¼	4 ft 3¾ in	16,000	203,000	11	140	160
	1⅝	3 ft. 10¼ in	26,000	343,000	18	240	
	2	3 ft. 6 in	40,000	520,000	28	360	
5	1⅝	5 ft. 1⅜ in	26,000	343,000	18	240	275
	2	4 ft 8½ in.	40,000	520,000	28	360	
	2½	4 ft. 2 in.	63,000	813,000	44	565	
6	2	5 ft 11 in.	40,000	520,000	28	360	450
	2½	5 ft 4½ in.	63,000	813,000	44	565	
	3	4 ft. 10 in.	90,000	1,170,000	63	810	
8	2¾	7 ft. 6¼ in.	76,000	983,000	53	680	700
	3¼	6 ft. 11¾ in.	106,000	1,373,000	74	950	
	4	6 ft. 2 in.	160,000	2,080,000	110	1,440	

Directions for installing the Venturi meter tube are given by the Builders Iron Foundry as follows:

"The Meter Tube is set in the pipe line in the same manner as ordinary pipe, the shorter cone forming the inlet, or upstream end. A notch in the edge of each flange denotes the top. It is not essential that the tube be horizontal; it can be inclined or vertical.

A straight length of pipe of the same diameter as the meter tube should immediately precede the inlet and contain no gate valve or other fitting liable to disturb the smooth flow of the water. The length of this pipe should be at least six times the diameter of the tube for sizes up to 24 in. and at least 12 ft. for larger sizes. If the outlet end of the meter tube is of different diameter from the pipe line an increaser or decreaser should be placed at this point. It is unnecessary to have a straight length of pipe on the outlet side of the meter tube.

For standard installations both meter and instrument should be set at a point where the working pressure is at least 12 lb. per square inch. Frequently, however, this requirement may be modified after consultation with our Engineering Department.

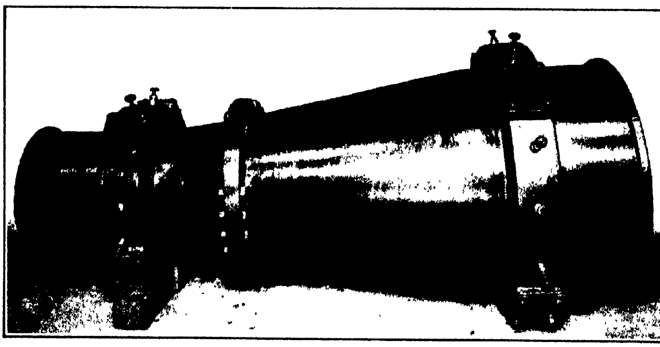


Fig. 48.—Inlet cone and throat of large venturi meter for sewage.

Two small pressure pipes connect the meter tube with the instrument. These can be brass, lead, lead-lined or other non-corrosive pipe, 3/4 in. diameter if the length is 50 ft.; 1 in. diameter if the length is 100 ft., etc., and connection should be made at the side of each pressure chamber. The piping should have a pronounced up or down grade, contain no summits or depressions where air or silt might collect, and a valve (or corporation cock) should be placed on each pressure pipe close to the meter tube. If a summit or depression is absolutely unavoidable, a blow-off valve should be provided at such point. All joints must be perfectly tight and the piping properly protected from frost."

The foregoing paragraphs relate especially to the use of the Venturi meter for measuring water. The principle is exactly the same when

sewage is to be measured, but on account of the suspended matter in the sewage, which might clog the tubes and interrupt the operation of the register, it becomes necessary to adopt special precautions or use a somewhat different pattern of instrument. Fig. 48 shows the inlet cone and throat of a large meter especially constructed for measuring sewage.

It will be noted that at each annular chamber or piezometer ring there are valves by which the pressure openings can be closed, and these valves are so designed that in closing a rod is forced through the opening so as to clean out effectually any matter which may have clogged it. When all four of these valves have been closed the plates covering the

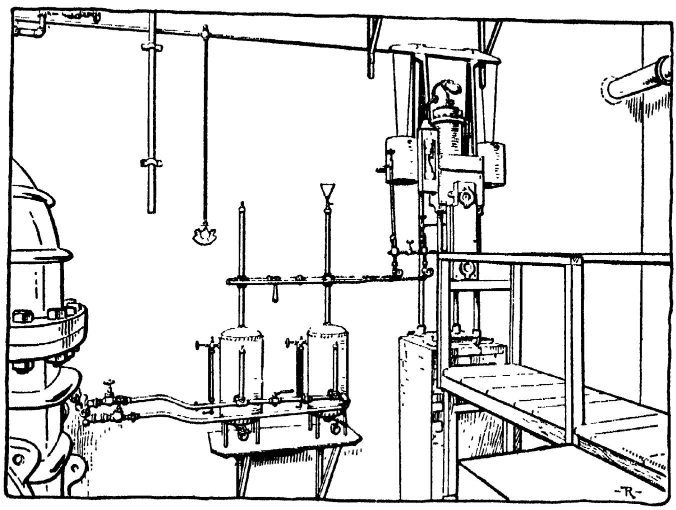


FIG. 49.—Oil seal and register, Ward Street Pumping Station, Boston.

hand holes in the pressure chamber may be removed and the chamber cleaned by flushing with hose or otherwise.

In order to prevent the interference with the operation of the register, by clogging, an oil seal is inserted in the pressure pipe, between the meter tube and the register. The pressure is transmitted as far as the seal, through water in the pressure pipes, and from the seal to the register through oil. Thus it is impossible for any sewage to get into the register and interfere with its proper operation. Such an oil seal is shown in Fig. 49, which illustrates the apparatus at the Ward Street Pumping Station of the Metropolitan Sewerage Works.

Float measurements of the flow in sewers are rarely made except in

rectangular channels or for the approximate determination of the velocity of flow between two manholes; but in studies of tidal currents or of sewage currents in bodies of water into which sewage may be discharged, floats are universally employed.

Three types of floats may be used—surface floats, subsurface floats, and rod or spar floats. Only surface velocities can be obtained by the use of surface floats and the results can be considered only as approximations, owing to the modifying effects of the wind. Subsurface floats consist of relatively large bodies slightly heavier than water, connected by fine wires to surface floats of sufficient size to furnish the necessary flotation and carrying markers by which their courses may be traced. The resistance of the upper float and connecting wire is generally so slight that the combination may be assumed to move with the velocity of the water at the position of the submerged float. Rod floats have been used for measuring flow in open flumes, with a high degree of accuracy. They generally consist of metal cylinders so loaded as to float vertically. The velocity of the rod has been found to correspond very closely with the mean velocity of the water in the course followed by the float. Detailed descriptions of the methods of making accurate measurements of flow in rectangular flumes may be found in Francis' "Lowell Hydraulic Experiments" and in Hughes and Safford's "Hydraulics."

Current meter measurements may be employed for the accurate determination of the velocity of flow in sewers of considerable size or in open channels, provided there be not too much paper or other suspended matter likely to clog the meter. The current meter must be calibrated by moving it at a uniform speed in still water. Knowing the constants or rating of the meter, the average velocity of the water at the point where it is held may be obtained with a high degree of accuracy.

Gagings of flow may be made by several methods, the one-point method, the two-point method, the multiple-point method, the method of integrating in sections, and the method of integrating in one operation.

In the single-point method, the meter is usually held at 0.6 of the depth and in the center of the stream, and the result is assumed to indicate the mean velocity of the stream. This is but a rough approximation, suitable only for hasty observations with no pretense to accuracy.

In the two-point method, the velocity is observed at 0.2 and 0.8 of the depth, and the average of these two figures is taken to represent the average velocity in the vertical section. The stream can be divided into a number of vertical sections, and the average velocity in each determined approximately by this method.

By the multiple-point method, the velocity at each of a large number of points, each representing the center of an equal area of the cross-section of the stream, is determined, and the average of the observed velocities is then the mean velocity in the section. Or, the velocities are

observed at a large number of points and lines of equal velocity in the cross-section are then drawn and measured by planimeter; by utilizing the method employed in computing mean elevation of a given area from a contour map the average velocity may be found. The employment of this method assumes a condition of steady flow, not only for the whole body of water but also for each filament, since it is obviously impossible to observe simultaneously the velocities at all points in the cross-section.

By the method of integrating in sections, the cross-section of the stream is divided into a number of vertical sections and the mean velocity in each is determined by lowering and raising the meter from top to bottom and back to the top of each section, at a uniform speed, for each observation. This is usually the most accurate and satisfactory method of making ordinary current meter gagings.

In integrating in one operation, the meter is lowered and raised as in integrating by sections, but at the same time is moved in a horizontal direction across the stream at a uniform rate. The result is intended to show the average velocity of the stream at one operation. With a skillful operator, results of a high degree of accuracy may be obtained by this method, and much more rapidly than by integrating in sections.

In a masonry conduit of regular form it is possible to make integrations in one operation by means of a track-board and pivoted sleeve, by which the meter is guided so as to pass over the entire area of the section of the stream, and if it is moved at a uniform speed, results of great accuracy may be obtained in this way. This method is employed in gaging the flow in the aqueducts of the [Boston] Metropolitan Water Works, and has been described in detail by Walter W. Patch in an article entitled "Measurement of the Flow of Water in the Sudbury and Cochituate Aqueducts," in *Eng. News*, June 12, 1902, p. 488.

An excellent discussion upon measurement of flow by meter observations will be found in Hughes and Safford's "Hydraulics," and in Hoyt and Grover's "River Discharge," 1908. The subject is also treated by John Clayton Hoyt and Nathan Clifford Grover in certain of the "Water Supply Papers" of the U. S. Geological Survey.

CHAPTER II

FLOW OF WATER IN PIPES AND CHANNELS

The science of hydrodynamics is that branch of hydraulics which treats of the mechanics of fluids in motion. The science of hydrostatics, on the other hand, treats of the mechanics of fluids at rest.

The term hydraulics is here used as having the broader significance including both hydrostatics and hydrodynamics. This chapter, therefore, embraces a brief reference to water and some of its more important physical attributes, and to certain of the principles of hydrostatics, and a more extended discussion of hydrodynamics or the principles governing flow, more particularly in sewers.

As sewage is composed of 99.8 per cent. of water and but 0.2 per cent. of mineral and organic matter, and has a specific gravity but very little in excess of unity (1.002 approximately), it is treated in hydraulic discussions as if it were clear water. The retarding effects of its contents at times and under certain conditions, and more particularly at the dead ends of the collecting system, are not to be lost sight of, however.

WATER

Water (H_2O) is a colorless liquid with high solvent powers. Having great fluidity, or little viscosity, it transmits pressures equally in all directions throughout its mass, the direction of the pressure being normal to the surface to which it is applied (Pascal's law).

Water may be assumed to be substantially incompressible in hydraulic computations, its coefficient of compressibility, or decrease in unit volume, caused by a pressure of one atmosphere (14.7 lb. per square inch), being approximately 0.00005. Its modulus of elasticity, E, in compression is approximately 296,000 lb. per square inch. The modulus increases and the coefficient of compressibility decreases slightly with increase in temperature. As an increase in pressure of 10 atmospheres increases the weight of water only by about 0.03 lb. per cubic foot, the effect of compressibility is negligible.

Molecular Changes.—Water reaches its maximum density at 39.3° F., at which point its specific gravity is unity. Water freezes at 32°

F., when its specific gravity is 0.99987. If it is absolutely quiescent, however, the temperature may fall to 20° F. or less before freezing takes place, and if on the other hand it is flowing rapidly, as in a stream, it will also fall in temperature considerably below 32° F. before freezing. Ice melts, however, at 32° F. or 0° C. It is owing to the fact that the maximum density of water occurs at a slightly higher temperature than the freezing point that bodies of fresh water do not freeze to a greater depth, for as the temperature of the water gradually falls in the early winter, the point of maximum density is reached at 39.3° F., and as the water chills further at the surface, by reason of its contact with the colder atmosphere, its specific gravity is raised and the cold layer of water therefore floats, except as wind currents may cause circulation and carry some of it to lower depths, and thus continues to fall in temperature until the ice sheet forms.

Water boils at sea level (barometric pressure of 30 in. of mercury, or 34 ft. of water) at 212° F., when its specific gravity is approximately 0.95865.

Weight of Water.—Fresh water weighs about 62.43 lb. per cubic foot. For approximate computations, the unit 62.5 lb. is often used for its convenience, as then

$$1 \text{ cu. ft.} = 62.5 \text{ lb.} = \frac{1000}{16} \text{ lb.} = 1000 \text{ oz.}$$

Salt water varies in density and weight, that of the Atlantic Ocean weighing, in the latitude of New York, approximately 64.1 lb., in the Gulf of Mexico, 63.9. The water in Great Salt Lake weighs from 69 to 76 lb. per cubic foot.

Ice weighs 57.2 to 57.5 lb. per cubic foot.

Sewage is usually assumed to have the same weight as water. In an investigation made by Harrison P. Eddy of the weight of the sewage discharged through the North Metropolitan Sewer at East Boston, a specific gravity of 1.0018 was found, the sewage having 1022 parts of chlorine per million. This would correspond to an excess of 0.1 lb. per cubic foot, over the weight of fresh water, and this was a fairly strong American sewage, containing much salt or sea water.

The Atmospheric Pressure at sea level will sustain a column of mercury 30 in. high, in vacuum, and of water, 34 ft. As mercury weighs 0.49 lb. per cubic inch this corresponds to 30x0.49 = 14.70 lb. per square inch pressure (1.033 kg. per square centimeter). This is known as the pressure of one atmosphere, the pressure of two atmospheres being double this amount, or approximately 29.4 lb. per square inch.

TABLE 5.—ATMOSPHERIC PRESSURES AND EQUIVALENTS

(Merriman's "Treatise on Hydraulics, 1912," p. 8)

Mercury barometer, inches	Pressure pounds per square inch	Pressure atmospheres	Water barometer, feet	Elevations, feet	Boiling point of water, Fahrenheit
31	15.2	1.03	35.1	−890	213.9°
30	14.7	1.00	34.0	0	212.2
29	14.2	0.97	32.9	+920	210.4
28	13.7	0.93	31.7	1,880	208.7
27	13.2	0.90	30.6	2,870	206.9
26	12.7	0.86	29.5	3,900	205.0
25	12.2	0.83	28.3	4,970	203.1
24	11.7	0.80	27.2	6,080	201.1
23	11.3	0.76	26.1	7,240	199.0
22	10.8	0.72	24.9	8,455	196.9
21	10.3	0.69	23.8	9,720	194.7
20	9.8	0.67	22.7	11,050	192.4

The Acceleration due to Gravity is approximately 32.16 ft. per second at sea level. Hering gives in his "Conversion Tables" the following values at sea level and 45° latitude for the linear acceleration due to

	Logarithm.
Gravity = 980.5966 cm. per sec. (Aprx. 1000).	2.9914904
= 35.3015 km. per hr. per sec. (or per sec. per hr.) (Aprx. $\frac{7}{2}$x10)	1.5477929
= 32.1717 ft. per sec. per sec. (Aprx. 32)	1.5074746
= 21.9353 miles per hr. per sec. (or per sec. per hr.) (Aprx. 22.)	1.3411433
= 9.805966 meters per sec. per sec. (Aprx. 10). . . .	0.9914904

TABLE 6.—FUNCTIONS OF ACCELERATION DUE TO GRAVITY, g

(Hughes & Safford's "Hydraulics," p. 8)

	In feet		In meters	
	Number	Log	Number	Log
g	32.16	1.5073	9.803	0.9914
$2g$	64.32	1.8083	19.607	1.2924
$(2g)^{\frac{1}{2}}$	8.02	0.9042	4.428	0.6462
$\frac{2}{3}(2g)^{\frac{1}{2}}$.	5.347	0.7281	2.952	0.4701
$\frac{1}{2g}$	0.01555	$\bar{2}.1917$	0.051	$\bar{2}.7076$

Merriman credits Pierce with the following partly theoretical, partly empirical formula for the determination of the acceleration of gravity, g, in feet-per-second per second, for a latitude, l, and an elevation, e, in feet above the sea level,

$$g = 32.0894\ (1 + 0.0052375 \sin^2 l)\ (1 - 0.0000000957e)$$

Intensity of Water Pressure.—Ignoring the influence of changes in atmospheric conditions and external forces, the intensity of pressure on the unit of area, resulting from a column of fluid of given height, is equal to the weight of the fluid, per unit volume, times its height.

$$P = wh$$
$$P = \text{pounds per square foot}, = 62.4h$$
$$p = \text{pounds per square inch}, = 0.4333h$$
$$h = \frac{P}{w}$$
$$h = 0.016P \text{ in pounds per square foot}$$
$$h = 2.308p \text{ in pounds per square inch}$$

where w = weight of water per cubic foot and h = head or height of column of water, in feet.

Expressed in words, this means that a pressure of 1 lb. per square inch corresponds to a head of 2.308 ft. of water. A pressure of 1 kg. per square centimeter corresponds to a head of 10 m.

A head of 1 ft. of water produces a pressure of 0.433 lb. per square inch. A head of 1 m. produces a pressure of 0.1 kg. per square centimeter.

TABLE 7.—CONVERSION FACTORS FOR UNITS OF PRESSURE

(Hughes & Safford's "Hydraulics," p. 6)

	Feet of water	Log	Inches of mercury	Log	Pounds per square inch	Log	Pounds per square foot	Log
Pounds per square inch to....	2.308	0.3632	2.037	0.3090	1.0000	0.0000	144.00	2.1584
Pounds per square foot to .	0.01603	2.2018	0.01414	2.1506	0.00694	3.8416	1.000	0.0000
Inches in height of mercury to . .	1.133	0.0542	1.000	0.0000	0.4910	1.6910	70.699	1.8494
Feet in height of fresh water to .	1.000	0.0000	0.8826	1.9458	0.4333	1.6368	62.4	1.7952
Feet in height of sea water to. .	1.025	0.0107	0.9047	1.9565	0.4442	1.6475	64.0	1.8062
Atmospheres to	33.923	1.5305	29.942	1.4763	14.70	1.1673	2116.8	3.3257
Atmospheres to	Sea water 33.096							

Specific gravities used in this table are: distilled water, 1.000; sea water, 1.025; mercury 13.5956.

For rough calculations the weight of fresh water is frequently taken as 62.5 lb. per cubic foot; and one atmosphere equivalent to 34 ft. of fresh water, 33 ft. of sea water, or 30 in. of mercury.

THE FLOW OF WATER

The laws of hydraulics are essentially similar to the fundamental laws of mechanics. The basic principles governing the flow of water,

neglecting the disturbing or modifying influences of friction and initial pressure, are founded upon the laws of falling bodies.

In 1643 Torricelli enunciated the theorem that, "the velocity of a fluid passing through an orifice in the side of a reservoir is the same as that which is acquired by a body falling freely in vacuo from a vertical height measured from the surface of the fluid in the reservoir to the center of the orifice." (Hughes and Safford's "Hydraulics," page 8).

In 1738 Daniel Bernoulli, the eminent mathematician of Basle, Switzerland, propounded the important hydraulic law of the conservation of energy in fluids, which may be stated thus: "At every section of a continuous and steady stream of frictionless fluid, the total energy is constant; whatever energy is lost as pressure is gained as velocity. Therefore, in terms of head: Total energy = velocity head + pressure head + head due to position = constant." (Hughes and Safford's "Hydraulics," page 81.)

Laws of Falling Bodies.—Neglecting the influence of friction, the laws of falling bodies are as follows:

If v = velocity in feet per second at any moment,
t = time in seconds,
h = fall or vertical distance traveled, in feet,
g = acceleration of gravity (32.16 approximately).

$$v = gt = 32.16t \qquad (1)$$

or, in words, the velocity of a falling body in a vacuum at any moment is equal to the time of the fall multiplied by the acceleration of gravity.

$$h = \frac{v}{2}t = \tfrac{1}{2}gt^2 = \frac{32.16}{2}t^2 = 16.08t^2 \qquad (2)$$

or, the distance traversed, or the fall in feet, is equal to one-half of the product of the acceleration of gravity times the square of the time, in seconds, elapsing in the fall.

$$h = \frac{v^2}{2g} = \frac{v^2}{64.32} = 0.01555v^2 \qquad (3)$$

or the distance traversed, or the fall in feet, or the velocity head, is equal to the square of the velocity divided by two times the acceleration of gravity.

$$v = \sqrt{2gh} = 8.02\sqrt{h} \qquad (4)$$

or the velocity is equal to the square root of two times the fall in feet, multiplied by the acceleration of gravity. The velocity then varies as the time and as the square root of the head, and the head varies as the square of the time and the square of the velocity.

If there be an initial velocity, V, in feet per second,

Equation (1) becomes	$v = V + gt$	(5)
Equation (2) becomes	$h = \frac{1}{2}gt^2 \pm Vt$	(6)
Equation (4) becomes	$v = \sqrt{2gh} \pm V$	(7)

TABLE 8—THEORETICAL VELOCITY OF WATER IN FEET PER SECOND FOR VARIOUS HEADS

$V = \sqrt{2gh}$, $g = 32.16$ (U. S. Reclamation Service)

Head in feet	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.0	2.5	3.6	4.4	5.1	5.7	6.2	6.7	7.2	7.6
1	8.0	8.4	8.8	9.1	9.5	9.8	10.1	10.5	10.8	11.1
2	11.3	11.6	11.9	12.2	12.4	12.7	12.9	13.2	13.4	13.7
3	13.9	14.1	14.3	14.6	14.8	15.0	15.2	15.4	15.6	15.8
4	16.0	16.2	16.4	16.6	16.8	17.0	17.2	17.4	17.6	17.8
5	17.9	18.1	18.3	18.5	18.6	18.8	19.0	19.2	19.3	19.5
6	19.6	19.8	20.0	20.1	20.3	20.5	20.6	20.8	20.9	21.1
7	21.2	21.4	21.5	21.7	21.8	22.0	22.1	22.3	22.4	22.5
8	22.7	22.8	23.0	23.1	23.3	23.4	23.5	23.7	23.8	23.9
9	24.1	24.2	24.3	24.5	24.6	24.7	24.8	25.0	25.1	25.2
10	25.4	25.5	25.6	25.7	25.9	26.0	26.1	26.2	26.4	26.5
11	26.6	26.7	26.8	27.0	27.1	27.2	27.3	27.4	27.5	27.7
12	27.8	27.9	28.0	28.1	28.2	28.4	28.5	28.6	28.7	28.8
13	28.9	29.0	29.1	29.2	29.4	29.5	29.6	29.7	29.8	29.9
14	30.0	30.1	30.2	30.3	30.4	30.5	30.6	30.7	30.9	31.0
15	31.1	31.2	31.3	31.4	31.5	31.6	31.7	31.8	31.9	32.0
16	32.1	32.2	32.3	32.4	32.5	32.6	32.7	32.8	32.9	33.0
17	33.1	33.2	33.3	33.4	33.5	33.5	33.6	33.7	33.8	33.9
18	34.0	34.1	34.2	34.3	34.4	34.5	34.6	34.7	34.8	34.9
19	35.0	35.0	35.1	35.2	35.3	35.4	35.5	35.6	35.7	35.8
20	35.9	36.0	36.0	36.1	36.2	36.3	36.4	36.5	36.6	36.7
21	36.8	36.8	36.9	37.0	37.1	37.2	37.3	37.4	37.4	37.5
22	37.6	37.7	37.8	37.9	38.0	38.0	38.1	38.2	38.3	38.4
23	38.5	38.5	38.6	38.7	38.8	38.9	39.0	39.0	39.1	39.2
24	39.3	39.4	39.5	39.5	39.6	39.7	39.8	39.9	39.9	40.0
25	40.1	40.2	40.3	40.3	40.4	40.5	40.6	40.7	40.7	40.8
26	40.9	41.0	41.1	41.1	41.2	41.3	41.4	41.4	41.5	41.6
27	41.7	41.8	41.8	41.9	42.0	42.1	42.1	42.2	42.3	42.4
28	42.4	42.5	42.6	42.7	42.7	42.8	42.9	43.0	43.1	43.2
29	43.2	43.3	43.3	43.4	43.5	43.6	43.6	43.7	43.8	43.9
30	43.9	44.0	44.1	44.2	44.2	44.3	44.4	44.4	44.5	44.6
31	44.7	44.7	44.8	44.9	44.9	45.0	45.1	45.2	45.2	45.3
32	45.4	45.4	45.5	45.6	45.6	45.7	45.8	45.9	45.9	46.0
33	46.1	46.1	46.2	46.3	46.3	46.4	46.5	46.6	46.6	46.7
34	46.8	46.8	46.9	47.0	47.0	47.1	47.2	47.2	47.3	47.4
35	47.4	47.5	47.6	47.6	47.7	47.8	47.9	47.9	48.0	48.1
36	48.1	48.2	48.3	48.3	48.4	48.5	48.5	48.6	48.6	48.7
37	48.8	48.8	48.9	49.0	49.1	49.1	49.2	49.2	49.3	49.4
38	49.4	49.5	49.6	49.6	49.7	49.8	49.8	49.9	50.0	50.0
39	50.1	50.1	50.2	50.3	50.3	50.4	50.5	50.5	50.6	50.7
40	50.7	50.8	50.8	50.9	51.0	51.0	51.1	51.2	51.2	51.3
41	51.4	51.4	51.5	51.5	51.6	51.7	51.7	51.8	51.9	51.9
42	52.0	52.0	52.1	52.2	52.2	52.3	52.3	52.4	52.5	52.5
43	52.6	52.7	52.7	52.8	52.8	52.9	53.0	53.0	53.1	53.1
44	53.2	53.3	53.3	53.4	53.4	53.5	53.6	53.6	53.7	53.7
45	53.8	53.9	53.9	54.0	54.0	54.1	54.2	54.2	54.3	54.3
46	54.4	54.5	54.5	54.6	54.6	54.7	54.7	54.8	54.9	54.9
47	55.0	55.0	55.1	55.2	55.2	55.3	55.3	55.4	55.5	55.5
48	55.6	55.6	55.7	55.7	55.8	55.9	55.9	56.0	56.0	56.1
49	56.1	56.2	56.3	56.3	56.4	56.4	56.5	56.5	56.6	56.7

FLOW OF WATER THROUGH PIPES

Water seeks its own level, the level or surface being approximately perpendicular to the direction of the force of gravity. Conversely, if its surface be not level, it will flow from the higher level to the lower. This is but another way of saying that difference in pressure, or in level or "head," as it is called technically is necessary to make water flow—a fact sometimes overlooked.

If, then, there be available a certain difference in level—called "fall" if measured from the upper point to the lower, or "head" if measured from the lower to the upper—between two points along a pipe, conduit, or channel carrying water or any other liquid, flow will be induced at a rate dependent, first, upon the fall as compared with the length traversed; second, upon the cross-section of the pipe, conduit, or channel; third, upon the character of its interior surface; fourth, upon the condition of flow with reference to the pipe, i.e., whether the pipe is under pressure or not, whether it is flowing full or partly full, and whether it is flowing uniformly, steadily, variably, or intermittently on account of constant or variable cross-section, or other cause; and, fifth, upon the character, specific gravity and viscosity of the liquid.

Let us examine briefly the hydraulic conditions of flow, first, in pipes flowing full or under pressure, i.e., in pipes in which the pressure is outward, as in water pipes, and second, in pipes or conduits flowing barely full or partly full in which there is no outward pressure of the liquid in all directions, or in which the pressure may be said to be inward, as in the case of sewer pipes.

Bernoulli's theorem, that the total energy in a steady stream of frictionless fluid is a constant and is equal to the elevation plus the velocity head plus the pressure head, may be expressed by the following formula:

$$H = h_e + h_v + h_p = h_e + \frac{v^2}{2g} + \frac{p}{w}$$

where H = total head,
h_e = the height of any point above any assumed plane of reference, the reference plane,
h_v = velocity head,
h_p = pressure head,
p = pressure in pounds per unit area,
w = weight of water per unit volume,
v = velocity of flowing particles, in unit of distance, per second.

Practically the conditions of the perfect fluid do not exist, and another element enters the problem, the frictional resistance of the pipe, channel, or conduit to the flowing fluid. This factor is covered in Bernoulli's theorem by the addition of another term in the equation

just given. As applied to two different points, A and B, upon the pipe-line:

$$H=h_e+h_v+h_p=h'_e+h'_v+h'_p+f,$$

using the same nomenclature and f being the loss in head due to the frictional resistance of the surface traversed by the fluid in passing from point A to point B.

The more important elements of frictional loss in pipes are the friction loss due to the interior surface of the pipe, the loss on entrance into the pipe, called the "entry head," losses due to sudden enlargement, losses due to sudden contraction, losses due to bends, losses due to gates, etc. In sewer construction, the loss due to the friction upon the interior of the pipe surface is practically the only one which has generally to be considered.

The entry head loss is approximately equal to 0.505 times the velocity head, or $0.5v^2/2g$, where the pipe enters from a manhole or reservoir of some sort. If the entry is made through a bell mouth, however, this loss may be reduced to less than 0.1 times the velocity head.

The loss due to sudden enlargement is equal to the square of the difference of the velocities in the two sections divided by $2g$, or $(v-v_1)^2 \div 2g$. If the enlargement is made gradual by a tapering connection, the loss due to enlargement may be reduced to a negligible quantity. The maximum discharge from a divergent mouthpiece is derived when the angle made by the sides of the mouthpiece is approximately 13° 24′.

A. P. Folwell, in an interesting article on "Lost Head in Water Supply Systems" (*Eng. News*, Apr. 17, 1902) recommends as sufficiently accurate for most practical purposes, the following approximate allowances for loss in head above that in an equal length of straight pipe:

In open valve, loss equal to that in 5 ft. of straight pipe, in excess of loss in equal length of straight pipe.

In half closed valve, 80 ft. ditto.

In ordinary cast iron 90° bends, 10 ft. ditto.

In ordinary tee, 3 ft.

In ordinary cross, 10 ft.

William O. Webber reported in *Eng. News*, Jan. 10. 1907, page 38, that the obstructions due to valves and fittings, expressed in the number of feet of clean, straight pipe of the same size which would cause the same loss as the fitting, were found by experiment to be as follows: 6-in. Pratt and Cady check valve, 50; 6-in. Walworth globe check valve, 200; 4-in. Pratt and Cady check valve, 25; 4-in. Walworth globe check valve, 130; 2½ to 8-in. long-turn ells, 4; 2½ to 8-in. short-turn ells, 9; 3 to 8-in. long-turn tees, 9; 3 to 8-in. short-turn tees, 17; one-eighth bends, 5.

Hydraulic Grade Line.—The hydraulic grade line represents the distance above a given plane of reference to which water would rise

at various points along any pipe-line or conduit, discharging under pressure, were piezometer tubes, or vertical pipes open to the atmosphere, inserted in the pipe-line. It is a measure of the pressure head available at these points. The hydraulic grade line will, of course, be influenced not only by the elevation of the points under question and the frictional resistance due to the rugosity of the internal pipe surface, but also by anything influencing the velocity head. In the case of a canal or open channel, in contradistinction to the pipe under pressure, the hydraulic grade line corresponds with the profile of its water surface.

Steady flow exists in a pipe-line, canal, or stream when equal quantities of water pass the same point in like intervals of time, or, in other words, when the discharge is constant for successive intervals of time.

Uniform flow exists when the cross-section and the mean velocity of the flowing stream are the same at every point. Uniform flow is a steady flow in which the cross-sections of the stream are all alike, and its surface is parallel to its bed.

The difference in these two conditions of flow must be clearly borne in mind on account of its bearing upon loss of head due to various causes. It is illustrated by the comparison in flow through a pipe line of uniform diameter throughout its length and through a Venturi meter the ends of which are of similar diameter. While both may be discharging the same quantity of water, the flow in the former is uniform, in the latter, steady, due to its varying cross-section.

Critical Velocity.—Hughes and Safford state (Hydraulics, pp. 90–92) "Turbulent eddying motion exists in nearly all cases in practical hydraulic problems, and the resistance to flow varies in proportion to some power of the mean velocity between 1.7 and 2.0 or more. Certain investigations, however, have shown that at very low velocities the motion of the water is in parallel stream lines, that is, without the disturbance due to eddying motion; and the resistance to flow varies nearly directly as the mean velocity of flow. The velocity at which turbulent eddying motion begins or ceases is called the critical velocity.

"Reynolds[1] made experiments to determine the point of critical velocity, and found that there were two critical values for any pipe or tube; 'one at which steady motion changed into eddies, the other at which eddies changed into steady motion.' The former change was found to occur at velocities considerably higher than the latter; and the two critical points are, therefore, called 'the higher critical velocity' and 'the lower critical velocity.'

"For the higher critical velocity,

$$v_c = \frac{1}{43.79} \cdot \frac{P}{D} \text{ (meters per second), or}$$

$$v_c = 0.2458 \frac{P}{D} \text{ (feet per second),}$$

[1] Osborne Reynolds, Phil. Trans. of the Roy. Soc., 1883, pp. 935, et seq.

where D = the diameter of the pipe in meters, or feet,
$P = (1 + 0.0336T + 0.000221T^2)^{-1}$ is the temperature correction,
T = temperature of the water, degrees Centigrade.

For the lower critical velocity,

$$v_c = \frac{1}{278}\frac{P}{D} \text{ (meters); or } v_c = 0.0387\frac{P}{D} \text{ (feet).}$$

"Experiments by Barnes and Coker[1] show values for the higher critical velocity fully double those of Reynolds, and for the lower critical velocity as little as half as much as Reynolds.

"All these experiments showed that disturbances in the supply tank, or jarring of the pipes, made a marked change in the point of critical velocity. For practical conditions the point of critical velocity cannot be very precisely determined; and except for small pipes is usually too low to be considered.

(Williams, Hubbell and Fenkell's discussion on "Flow of Water in Pipes," Trans. Am. Soc. C. E., April, 1902, p. 367).

"* * * * * * The experiments of Poiseuille, Hagen, Jacobson and Hazen show that when water flows through capillary tubes or fine sands where it is prevented from taking up internal motions, because the area of the cross-section of the stream is almost molecular, that H_f varies very nearly as the first power of V. All reliable experiments on record show that as the diameter decreases the exponent of V, in $H_f = mV^n$, decreases, as has been shown for the lines investigated in this paper: 30 in., $H_f = mV^2$; 16 in., $H_f = mV^{1.86}$; 12 in., $H_f = mV^{1.78}$; and 2 in. brass, $H_f = mV^{1.76}$, from a possible limit of V^2. In other words, the more the chance for internal resistance, the higher the exponent of V. To the writers, then, the variation of the exponent of V is an index of the character of the flow, and when that becomes greater than unity, straight-line flow is over, or, the critical velocity of Professor Reynolds is past. If, then, these internal motions are capable of so increasing the rate of loss of head, it is evident that in them the controlling conditions of the laws of flow are to be looked for, rather than in the surface resistances. But, beyond this first critical velocity, there appear to be others where peculiar phenomena appear, * * * * *."

The resistance to flow for velocities under the critical velocity [the points at which eddying begins and ends] for capillary tubes and small pipes may be approximately computed by the following formula suggested by Allen Hazen:[2]

$$V = cSD_i^2\left(\frac{t+10}{60}\right)$$

S = the slope of the hydraulic grade line,
V = the mean velocity of flow in feet per second,
D_i = the diameter in inches,

[1] Proc. of the Roy Soc., Vol. 74, pp. 341–356.
[2] Allen Hazen, Trans. Am. Soc. C. E., Vol. 51, pp. 316–319.

t = the temperature of the water, degrees Fahrenheit,
c = a factor; from Saph and Schoder's experiments on brass pipes Hazen determined c to be from 462 to 584; Williams and Hazen use a value of 475 in their "Hydraulic Tables."

DISCHARGE OF PIPES

Equation of Continuity.—The discharge of any pipe-line is given by the expression, $Q = AV$, in which

Q = the discharge in cubic feet per second,
A = the area of cross-section of the flowing stream in square feet,
V = velocity in feet per second, or other units of time and space.

If the flow is continuous in any given pipe-line there follows the equation of continuity,

$$AV = av$$

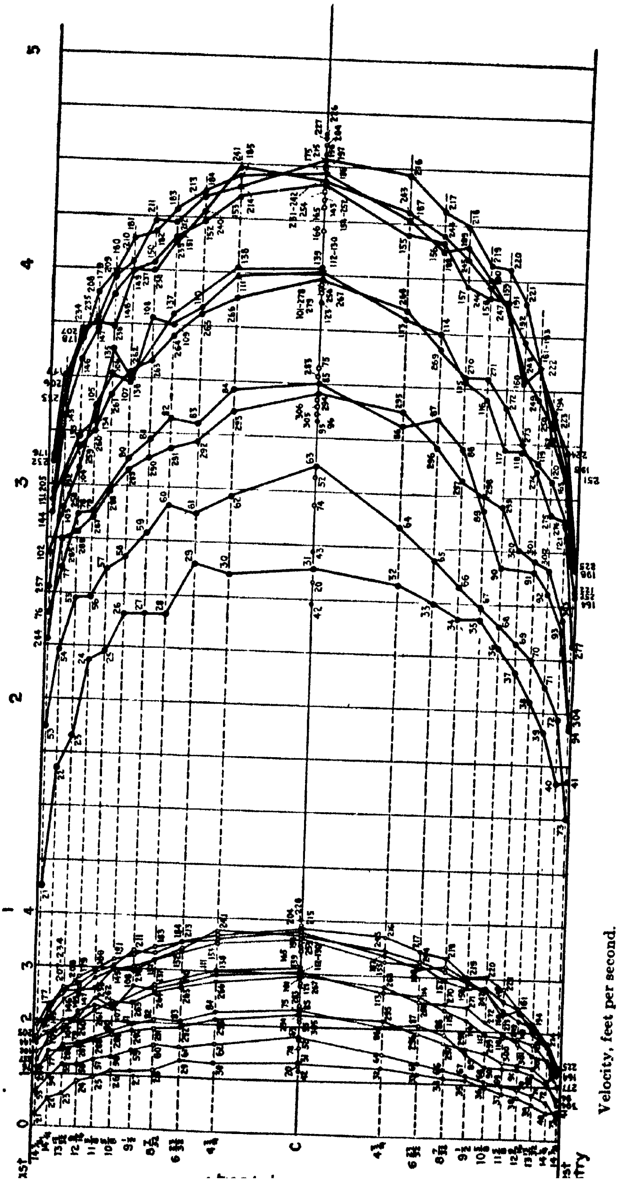
using the same nomenclature, the large letters referring to the area and the velocity of the flowing stream at one cross-section of the pipe-line, and the small letters to the area and velocity at another point.

In the above cases by the term "velocity" is meant the mean or average velocity in the entire cross-section. Where the term velocity in feet per second or in some other units of time and space, has been used, the mean velocity in the cross-section of the flowing stream has been referred to, for it is clear that with frictional resistance on the walls of the pipe or conduit, the velocity of flow at the point of contact of the fluid with these walls must be less than that in the center of the stream. The variation in the velocity at different points in the cross-section of any pipe discharging under pressure is shown in an approximate manner in Figs. 12, 13 and 19.

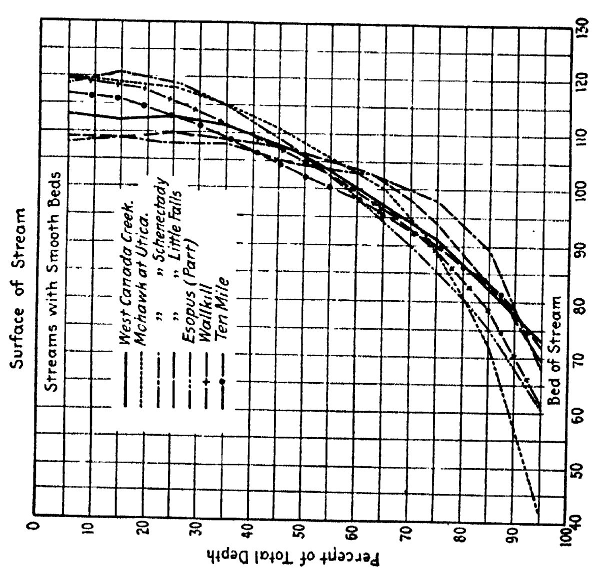
Mean velocity is dependent upon, first, the available head or fall, second, the resistance to the flowing stream.

The resistance in its turn varies with the length, wetted perimeter, and cross-section of the pipe, conduit or channel; the rugosity, or roughness, of its interior surface; the temperature and hence viscosity of the fluid; and the condition of flow, uniform, steady or variable. The resistance was shown by Dubuat to be independent of the water pressure, thus establishing the essential difference between the frictional resistance of a fluid and a solid as compared with the frictional resistance of two solids—the latter of which is dependent upon the weight or pressure of one solid upon the other.

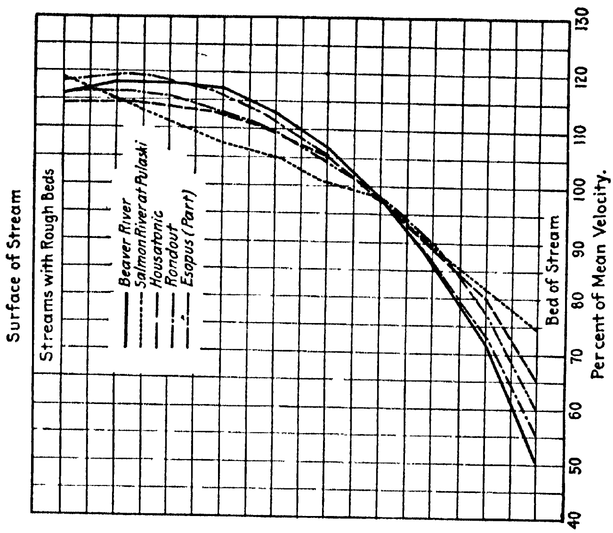
Development of Formulas for Flow in Pipes and Channels.—Ganguillet and Kutter ("Flow of Water in Rivers and Other Channels," translated by Hering and Trautwine, 1892) state,



FIGS. 12 AND 13.—Velocity curves in 30-inch pipe in Detroit.
From paper by Williams, Hubbell and Fenkell, "Transactions" American Society of Civil Engineers, Volume XLVII, page 44.



Streams with smooth beds.



Streams with rough beds.

FIGS. 14 AND 15.—Velocity curves of streams with smooth and rough beds.

From Report of State Engineer of New York, 1902, Supplement, and Water Supply Paper 76, U. S. Geological Survey.

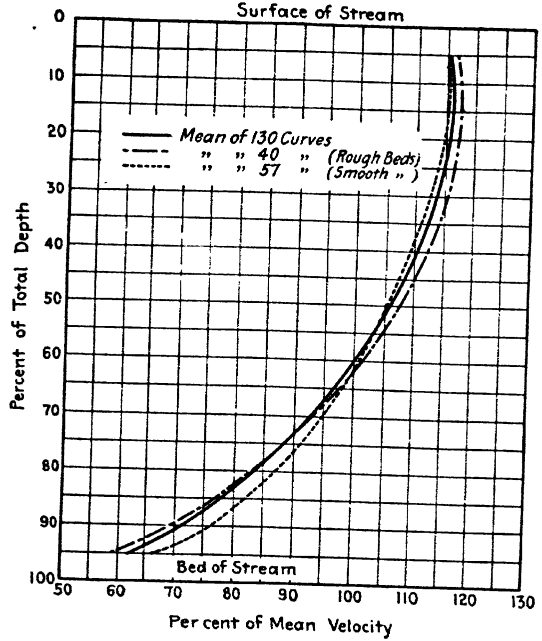


FIG. 16.—Mean velocity curves of New York and Connecticut streams.

From Report N. Y. State Eng., 1902, Supplement, and Water Supply Paper 76, U. S. Geological Survey.

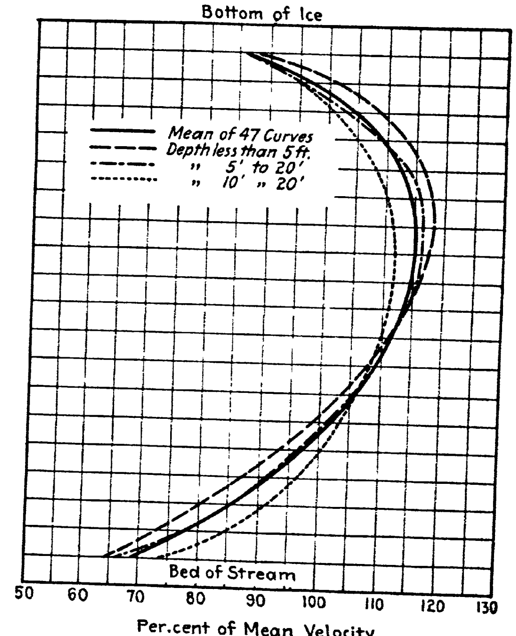


FIG. 17.—Mean velocity curves of streams with ice cover.

Refers to New York Streams, from Water Supply Paper 76, U. S. Geological Survey.

"The first attempt to discover the law by which the velocity of water depends upon the fall and the cross-section of the channel was, according to Hagen, made by Brahms (1753), who observed that the acceleration which we should expect in accordance with the law of gravity does not take place in streams, but that the water in them acquires a constant velocity. He points to the friction of the water against the wet perimeter as the force which opposes the acceleration, and assumes that its resistance is proportional to the mean radius R, i.e., to the area of cross-section divided by the wet perimeter.

Brahms and Chezy (1775) are to be regarded as the authors of the well-known formula

$$v=c\sqrt{\frac{a}{p}\times\frac{h}{l}}=c\sqrt{RS},$$

in which v = velocity in feet per second,

c = coefficient of roughness,

a = area of cross-section in square feet,

p = wetted perimeter in feet,

h = head or fall in feet,

l = length in feet,

R = hydraulic mean radius $=\frac{a}{p}$,

S = slope.

The principle established by Michelotti and Bossut, that the laws governing the flow of water must be established experimentally, led Dubuat (1779) to investigate the flow of the Canal du Jard and the River Heine in France, and of experimental channels. He concluded that the force producing flow was the fall or slope of the water surface of the flowing stream, and that the resistance must be equal to this accelerating force, under conditions of uniform flow.

De Prony concluded (Ganguillet and Kutter's "Flow of Water in Rivers and Other Channels," pp. 4 and 5).

"The particles of water in a vertical line in the cross-section of a stream move with different velocities, which diminish from the surface to the bottom.

The surface, bottom, and mean velocities stand in a certain relation to each other, which Dubuat, strange to say, finds to be independent of the size and form of the cross-section.

A layer of water adheres to the walls of the pipe or channel, and is therefore to be regarded as the wall proper which surrounds the flowing mass. According to Dubuat's experiments the adhesive attraction of the walls seems to cease at this layer, so that differences in the material of the walls produce no perceptible change in the resistance.

The particles of water attract each other mutually, and are themselves attracted by the walls of the channel. These attractions (resistances) may, in general, be expressed by means of two different values, which, however, are supposed to be the same nature and comparable with each other."

Later, H. Darcy, Inspector General of Roads and Bridges, noted that the pipes having the smoothest interior surface delivered the greatest quantity of water and thus indicated the least frictional resistance to the flow. Believing that similar conditions must prevail in flowing streams, he began a series of experiments, continued after his death by his assistant, the famous hydraulic engineer, H. Bazin. These experiments, which covered observations not alone upon canals and rivers but also upon artificial channels, led to the well-known Bazin formula for the flow of water in pipes.

Bazin's Old Formula for the flow of water is, in its general form, as follows:

$$RS = \left(\alpha + \frac{\beta}{R}\right)v^2,$$

or

$$v = \sqrt{\frac{RS}{\alpha + \frac{\beta}{R}}} = \sqrt{\frac{1}{\alpha + \frac{\beta}{R}}}\sqrt{RS}$$

The coefficients α and β were determined, graphically, from the plotted results of experimental measurements. Bazin grouped his channels under four classifications, for which he determined the values of α and β in Table 9 and to this classification a fifth was added in like manner by Ganguillet and Kutter, at a later date:

TABLE 9.—CONSTANTS FOR USE IN BAZIN'S (OLD) FORMULA

Category	Channels	α for English measure	α for metric measure	β
I	Cement Carefully planed wood	0.000046	0.00015	0.0000045
II	Smooth ashlar Brick Unplaned wood	0.000058	0.00019	0.0000133
III	Rubble masonry	0.000073	0.00024	0.0000600
IV	Earth	0.000085	0.00028	0.0003500
V	Carrying detritus and coarse gravel	0.000122	0.00040	0.0007000

The Chezy Formula.—This is

$$v = c\sqrt{RS}$$

in which v = mean velocity in feet per second,

R = hydraulic mean radius or area divided by wetted perimeter,

S = slope or ratio of fall to length,

c = coefficient varying with,

first, the roughness of the wetted perimeter decreasing with the increase in the roughness, most rapidly when R is small; second, with the value of the mean hydraulic radius, R, increasing with its increase, most rapidly when R is small; third, with the slope, S, decreasing with its increase in large

streams and increasing with its increase in small streams (Ganguillet and Kutter, p. 22).

The formula is essentially empirical in form but it has long remained the one most familiar to engineers, and as substantially all of the later results of experiments have been applied to it, as well as to some other formulas, the limits of its applicability have been better established than have those of any other formula for the flow of water in pipes, conduits, canals and rivers.

The determination of the coefficient c under different conditions has received much study from hydraulicians.

The Chezy formula may also be written in another form, which is attributed to Weisbach (see Coxe's translation of Weisbach's "Mechanics," p. 866).

$$h_f = f\frac{l}{d}\frac{V^2}{2g}$$

in which h_f = the head loss, in feet, in friction in the given length and diameter of pipe,

f = the coefficient of friction, which decreases with increase in pipe diameter and slightly with velocity of flow,

l = length of pipe, in feet,

d = internal diameter of pipe, in feet,

v = velocity of flow in pipe, in feet per second,

g = acceleration of gravity = 32.16.

These two forms of the Chezy formula have been arranged by Hughes and Safford ("Hydraulics," 1911, p. 285), as applied to the flow of water in pipes, in the following manner:

$$V = C(RS)^{\frac{1}{2}};\ \text{or}\ h_f = fLV^2/D2g$$

For uniform steady flow in circular pipes:

The mean hydraulic radius, $R = D/4$

The slope of the hydraulic grade line, $S = h_f/L$

The area of the stream, $A = \pi D^2/4$ Then:

The friction head, $h_f = \dfrac{4V^2L}{C^2D}$; or $h_f = f\dfrac{LV^2}{D2g}$

The mean velocity of flow in feet per second,

$$V = \frac{C}{2}\left(\frac{h_fD}{L}\right)^{\frac{1}{2}};\ \text{or}\ V = 8.02\left(\frac{h_fD}{fL}\right)^{\frac{1}{2}}$$

The discharge in cubic feet per second, $Q = AV = \dfrac{\pi D^2V}{4}$

$$Q = 0.3927\ C\left(\frac{h_fD^5}{L}\right)^{\frac{1}{2}};\ \text{or}\ Q = 6.3\left(\frac{h_fD^5}{fL}\right)^{\frac{1}{2}}$$

The diameter in feet required to deliver a given discharge,

$$D = 1.453\ \left(\frac{LQ^2}{h_fC^2}\right)^{\frac{1}{5}}\ \text{or}\ D = 0.479\left(\frac{fLQ^2}{h_f}\right)^{\frac{1}{5}}$$

Comparison of coefficients C and f,

$$C=\left(\frac{8g}{f}\right)^{\frac{1}{2}}=\frac{16.04}{(f)^{\frac{1}{2}}};\quad \text{and } f=\frac{8g}{C^2}=\frac{257.28}{C^2}$$

THE KUTTER FORMULA

Among the engineers who have given study to the correct determination of the coefficient c to be used in the Chezy formula for the flow of water in pipes, conduits, and channels, were the Swiss engineers, Ganguillet and Kutter, of Berne. Their results were first published in a series of articles in the German technical press. They were first translated into English by L. D'A. Jackson (London, 1876), and again by Rudolph Hering and J. C. Trautwine, Jr., in 1892, who presented them with additions in a volume entitled, "A General Formula for the Uniform Flow of Water in Rivers and Other Channels, by E. Ganguillet and W. R. Kutter, Translated from the German, With Numerous Additions including Tables and Diagrams, and the Elements of over 1200 Gagings or Rivers, Small Channels and Pipes," one of our American engineering classics.

In its general form,

$$v=\left(\frac{a+\frac{l}{n}+\frac{m}{S}}{1+\left(a+\frac{m}{S}\right)\frac{n}{R}}\right)\sqrt{RS}$$

The values a, l and m are constant and n varies with the degree of roughness. Substituting the numerical values found for the constants in metric measure, $a=23$, $l=1$, $m=0.00155$, we have in metric measure,

$$v=\left(\frac{23+\frac{1}{n}+\frac{0.00155}{S}}{1+\left(23+\frac{0.00155}{S}\right)\frac{n}{\sqrt{R}}}\right)\sqrt{RS}$$

in English measure,

$$v=\left(\frac{41.66+\frac{1.811}{n}+\frac{0.00281}{S}}{1+\left(41.66+\frac{0.00281}{S}\right)\frac{n}{\sqrt{R}}}\right)\sqrt{RS}$$

in which v = the mean velocity of the water,
R = the hydraulic mean radius,
S = slope of water surface per unit of length,
n = coefficient of roughness of the wetted perimeter.

For coefficients of roughness, n, with their reciprocals, etc., Ganguillet and Kutter suggested (p. 61): the values in Table 10.

TABLE 10.—VALUES OF n IN KUTTER FORMULA

	n	$\frac{1}{n}$	$a+\frac{1}{n}$
1. Channels lined with carefully planed boards or with smooth cement.	0.010	100.00	123
2. Channels lined with common boards	0.012	83.33	106
3. Channels lined with ashlar or with neatly jointed brickwork.	0.013	76.91	100
4. Channels in rubble masonry	0.017	58.82	82
5. Channels in earth, brooks and rivers	0.025	40.00	63
6. Streams with detritus or aquatic plants	0.030	33.33	56

In Hering and Trautwine's translation will be found in English measure, the results of 1200 experiments made in different places and countries up to that time, 1892. The more recent determinations of the coefficient of roughness n, for use in Kutter's formula, made chiefly upon sewer pipes, conduits and channels, have been summarized below.

Lowis D'A. Jackson's translation (1876) of "Kutter's Hydraulic Tables" cites the following values for use in Kutter's formula (p. 74):

0.009 Well-planed timber
0.010 Plaster in pure cement
0.011 Plaster in cement, with one-third sand
0.012 Unplaned timber
0.013 Ashlar and brickwork
0.015 Canvas lining on frames
0.017 Rubble
0.020 Canals in very firm gravel
0.025 Rivers and canals in perfect order and regimen, and perfectly free from stones and weeds.
0.030 Rivers and canals in moderately good order and regimen, having stones and weeds occasionally
0.035 Rivers and canals in bad order and regimen, overgrown with vegetation, and strewn with stones, or detritus of any sort.

Lowis D'A. Jackson made in 1877 and 1878, at the request of the Indian Government, an independent determination of a set of values of n. His figures were obtained from experiments on water works in South America and in Northern and Southern India and from official records in several other countries.

"Briefly, the results were, that none of the cases in canals in earth were below $n=0.017$, that the cases in which $n=0.025$ was approximately applicable were not canals in by any means perfect order, that any channels of a condition suited to $n=0.035$ were from irregularity beyond the scope of anything but excessively coarse and almost useless determination and that a large number of cases of canals in good order happen to give a value of n not far from 0.0225."

"Five fixed classes were therefore assigned to canals in earth of various soils, and in various conditions.

First $n = 0.020$ for very firm, regular, well-trimmed soil,

Second $n = 0.0225$ for firm earth, in condition above the average,

Third $n = 0.0250$ for ordinary earth in average condition,

Fourth $n = 0.0275$ for rather soft friable soil in condition, below the average,

Fifth $n = 0.030$ for rather damaged canals in a defective condition.

"The attempts of the author (Jackson) to determine independently values of n suited to canals in artificial materials, plank, rubble, ashlar, and cement, were ineffectual from want of sufficient mention of age, quality and condition of surface of these materials in recorded cases of experiment then forthcoming. For the special material, rubble, these latter did not afford quite sufficient reason for objecting to Herr Kutter's value of $n = 0.017$ for that material in a normal condition, but they did indicate a wide range of values; as to other materials, nothing resulted on account of the reason before given; the general conclusion was that each material should have a wider range of values of n suited to various conditions. Accepting, therefore, the normal values given by Herr Kutter as correct, the extension of their range was effected by the following arrangement.

$n = 0.010$; Smooth cement, worked plaster, planed wood, and glazed surfaces in perfect order.

$n = 0.013$; The materials mentioned under 0.010 when in imperfect or inferior condition. Also brickwork, ashlar, and unglazed stoneware in a good condition.

$n = 0.017$; Brickwork, ashlar, and stoneware in an inferior condition. Rubble in cement or plaster in good order.

$n = 0.020$; Rubble in cement in an inferior condition. Coarse rubble rough-set in a normal condition.

$n = 0.0225$; Coarse dry-set rubble in bad condition." (Jackson, "Hydraulic Manual.")

Major Allen Cunningham (1874–79) carried out experiments on a most extensive scale on the Upper Ganges Canal in India. The total number of velocity measurements was 50,000.

"After discussing various known formulas for mean velocity, the only ones that appeared worth extended trial were Bazin's formulas for the coefficients β and α, and Kutter's for the coefficient c. As to Bazin's two coefficients (β, α) the discussion shows that neither is reliable * * * * *. As to Kutter's coefficient c, the discrepancies between the 83 experimental and computed values were: thirteen over 10 per cent., five over $7\frac{1}{2}$ per cent., fifteen over 5 per cent., seventeen over 3 per cent., and thirty-three under 3 per cent.

"Now in all the discrepancies over 10 per cent., it was found that the state of water was unfavorable for the slope measurement. Taking this into account, along with the varied evidence in Kutter's work, it seems fair to accept Kutter's coefficient as of pretty general applicability; also that when the surface slope measurement is good, it will give results seldom exceeding

7½ per cent. error, provided that the rugosity coefficient of the formula is known for the site. For practical application extreme care would be necessary about the slope-measurement, and the rugosity coefficient could only be determined, according to present knowledge, by special preliminary experiments at each site.

"Much special experimenting was done (with surface slope measurement) and with the definite result that Kutter's formula was the only one not requiring velocity measurement of pretty general applicability, and would, under favorable conditions, give results differing by not more than 7½ per cent. from actual velocity measurements. This was surely a definite and important result." P. J. Flynn, "Irrigation Canals."

C. D. Smith reported before American Society of Irrigation Engineers (1894):

"I have found the coefficients of roughness in streams recently put in good order and regimen to vary from 0.020 to 0.0275, while if both coefficients were used in the same canal the difference in results would be over 40 per cent. Still engineers will usually use the coefficient 0.025 for all streams of this kind.

"Below I give a table of coefficients of roughness deduced from personal experience. Tests were made with current meter and weir. No difference has been designated between the experiments by the weir and meter measurements, for when comparisons were made, in similar streams, the results were found to be the same.

$n = 0.020$ firm soil trimmed with shovel.

$n = 0.021$ firm soil, the banks worn tolerably smooth, the soft dirt being worn off leaving surface of bank rather uneven.

$n = 0.022$ clay, with some loose gravel.

$n = 0.023$ clay, where velocity is not great enough to wear the banks smooth.

$n = 0.025$ new ditch in loam or clay, as usually left after completion when carefully constructed.

$n = 0.026$ banks sloping, with weeds occasionally along the banks.

$n = 0.0275$ pure sand uniform cross-section recently put in good order.

$n = 0.040$ grown up with weeds, in the center the weeds do not reach the surface.

$n = 0.045$ ditch in bad condition, grown up with weeds."

Theodore Horton (1901), in an admirable article upon "Flow in the Sewers of the North Metropolitan Sewerage System of Massachusetts" (Trans. Am. Soc. C. E., Dec., 1901, p. 78), gives an account of gagings made in the Metropolitan sewers with a current meter.

"The points selected for carrying out these observations were at manholes located some distance below the pumping stations, where the flow was free from any disturbing influence of the pumps. The points were about 800 ft. below the stations, in each case, and were far removed from any changes in alignment, cross-section or grade of the sewer. Below the East Boston pumping station the cross-section of the sewer is a 9-ft. circle of 12-in. brickwork, cement-washed, with a hydraulic gradient of 1 : 3000. Only one small local

connection enters this stretch of sewer. No changes in grade occur within a distance of 7000 ft. below the pumping station. At a point 2000 ft. below the pumping station, there is a change from a circular section to a horseshoe section of the same equivalent area. This section continues for a distance of 2000 ft. and then returns to a circular section. Below the Charlestown pumping station, the cross-section of the sewer is 6 ft. by 6 ft. 8 in., basket-handle, of 8-in. brickwork, cement-washed, with a hydraulic gradient of 1 : 2000. The cross-section and grade are uniform for a distance of about 5000 ft. below the pumping station, and no local connections enter the sewer within this distance."

The results of the test are shown in Table 11 and Fig. 18.

TABLE 11.—VALUES OF n IN KUTTER FORMULA DETERMINED FROM GAGINGS OF A CEMENT-WASHED BRICK TRUNK SEWER (HORTON)

No.	Depth	Q in cu ft per sec	Mean velocity	Hydraulic radius	C	n
Series of July, 1896—Charlestown Pumping Station						
I	1.02	8.60	1.99	0.688	107	0.0129
II	1.44	16.59	2.46	0.958	112	0.0131
III	1.91	26.81	2.82	1.187	115	0.0132
IV	2.40	38.82	3.13	1.387	118	0.0133
V	2.89	52.90	3.44	1.539	124	0.0130
Series of July, 1896—East Boston Pumping Station						
I	1.02	6.10	1.58	0.619	110	0.0122
II	1.52	15.67	2.21	0.928	126	0.0117
III	2.04	29.40	2.70	1.208	134	0.0116
IV	2.45	42.18	3.03	1.408	139	0.0115
V	3.16	69.50	3.48	1.830	141	0.0117
VI	3.75	94.60	3.73	1.999	145	0.0116
VII	4.62	138.00	4.18	2.309	150	0.0115
Series of December, 1897—Charlestown Pumping Station						
I	2.91	45.67	2.97	1.540	107	0.0149
II	3.29	56.14	3.16	1.650	111	0.0147
Series of November, 1897—East Boston Pumping Station						
I	2.15	30.13	2.55	1.280	123	0.0126
II	2.74	47.75	2.90	1.560	127	0.0127
III	3.19	62.05	3.06	1.762	126	0.0129
IV	3.20	64.82	3.18	1.771	131	0.0126
Series of June, 1900—Charlestown Pumping Station						
I	2.29	30.82	2.66	1.342	102	0.0151
II	2.78	41.39	2.86	1.508	104	0.0152
III	3.26	52.96	3.04	1.645	106	0.0152
Series of April, 1900—East Boston Pumping Station						
I	1.99	24.96	2.38	1.120	119	0.0130
II	2.83	48.26	2.82	1.606	121	0.0132
III	3.64	76.78	3.16	1.952	124	0.0133
IV	4.18	95.84	3.30	2.130	124	0.0136

Horton concluded among other things that the greatest change in internal surface of the sewers took place soon after the channels were put into operation, the initial coefficient of friction n, for use in Kutter's

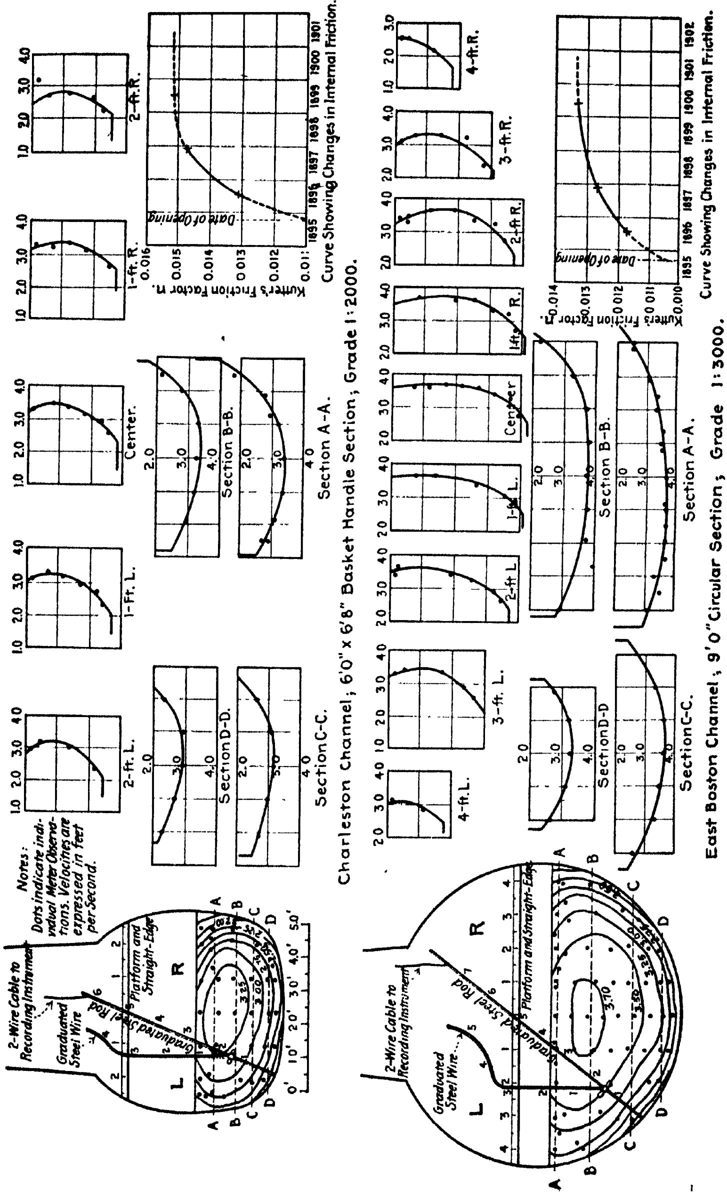


FIG. 18.—Velocity curves in two Metropolitan sewers (Boston).

formula being between 0.010 and 0.011, the Charlestown channel giving slightly the higher value. In comparing these changes in the values of n with the actual condition of the channels, it should be kept in mind that:

"The East Boston channel is of 3 ft. greater diameter than the Charlestown channel, that the invert of the East Boston channel is approximately 3 ft. above mean low water, while the Charlestown channel is 4 ft. below mean low water, and that the East Boston channel receives relatively less storm water than the Charlestown channel, and is, consequently, subject to less fluctuation of water surface. The importance of this last influence is evident from the fact that the deposit of both grease and organic growth appeared in greater abundance on the sides of the channels, and was greatest near the line of average flow. On the bottom of the channel there was practically no deposit; resulting, no doubt, from the scouring action of sand and other heavy particles transported along the invert by the sewage. This last feature is by no means novel, and has frequently been observed, though to a less extent, in water-supply conduits.

"The effect of the density of the sewage upon the carrying capacity of these channels appears to be slight, in view of the fact that the observations were made under all the varying conditions of storm and dry-weather flow. The possible effect of cleaning or scraping, however, might be much greater, but, at this date, no cleaning of any sort has taken place in these channels."

R. A. Hale, Arthur T. Safford, and Leonard Metcalf (1904) reported an approximate determination of c in the Chezy and n in the Kutter formula, as applied to extreme flood conditions upon the Merrimac River at Reed's Ferry, N. H., as developed from observed water levels at different nearby points upon the river. The discharge was approximately 63,760 cu. ft. per second; velocity, 3.96 ft. per second; R, 26.5; slope, 0.196 ft. per 1000 ft.; coefficient c, in Chezy formula, 55; coefficient n, in Kutter formula, 0.055. This result is not to be assumed as precise, but a fair approximation for such extreme flood conditions upon the Merrimac River.

C. C. Babb (1906) reported (*Eng. News*, Feb. 1, 1906) measurements made in an irrigation canal near Kimball, Alberta. The canal was in fairly good order, being free from weeds and the banks well preserved. The range of values found for n in five experiments was from 0.021 to 0.027, the average being 0.0236.

Freeman C. Coffin (1906) reported the coefficient of roughness which he found in the Cambridge, Mass., conduit—built under his direction in 1906, of Portland cement concrete, the surface of which had been subsequently brushed with neat cement—as being approximately $n = 0.011$ under the observed conditions of flow. These were, depth 1.75 ft. in circular conduit 63 in. in internal diameter, laid upon a slope of 3 in. per thousand feet.

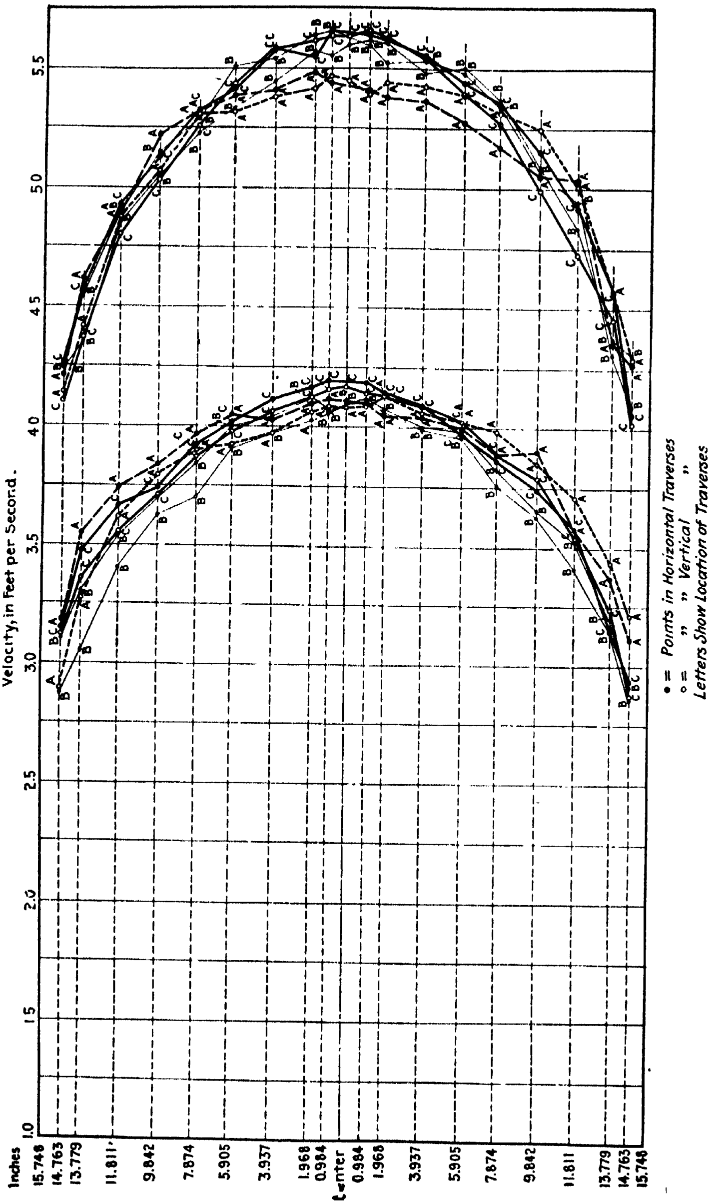


FIG. 19.—Distribution of velocities in 32-inch pipe. (Bazin.)
From Transactions of the American Society of Civil Engineers, Vol. XLVII, page 249

J. B. Lippincott (*Eng. News*, June 6, 1907) summed up the results of his experiments in canals in South California thus:

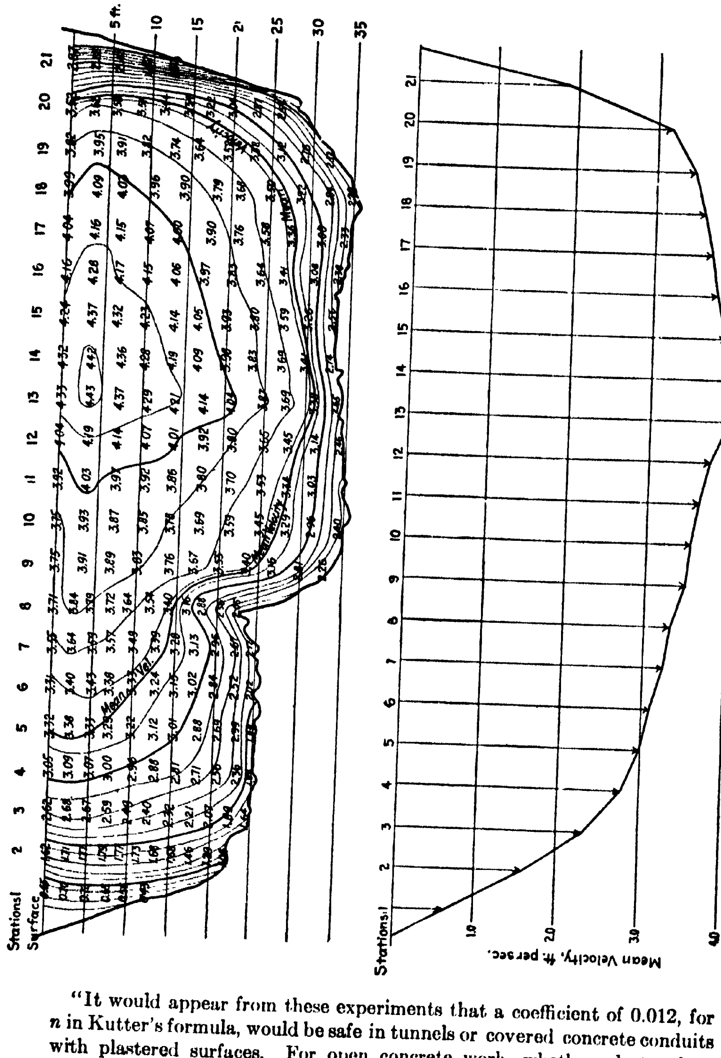


FIG. 20.—Lines of equal velocity and transverse curve of mean velocities, St. Clair River. From Annual Report of Chief of Engineers, U. S. A., 1900; Part 8, Appendix 3.

"It would appear from these experiments that a coefficient of 0.012, for n in Kutter's formula, would be safe in tunnels or covered concrete conduits with plastered surfaces. For open concrete work, whether plastered or not, where vegetation would occur, the value of n should be increased to

0.016 or 0.018. Where the grades of the conduit are so flat that velocities will be inadequate to keep the channel scoured, and under conditions where silt occurs in the water, a value of $n = 0.020$ or more should probably be used."

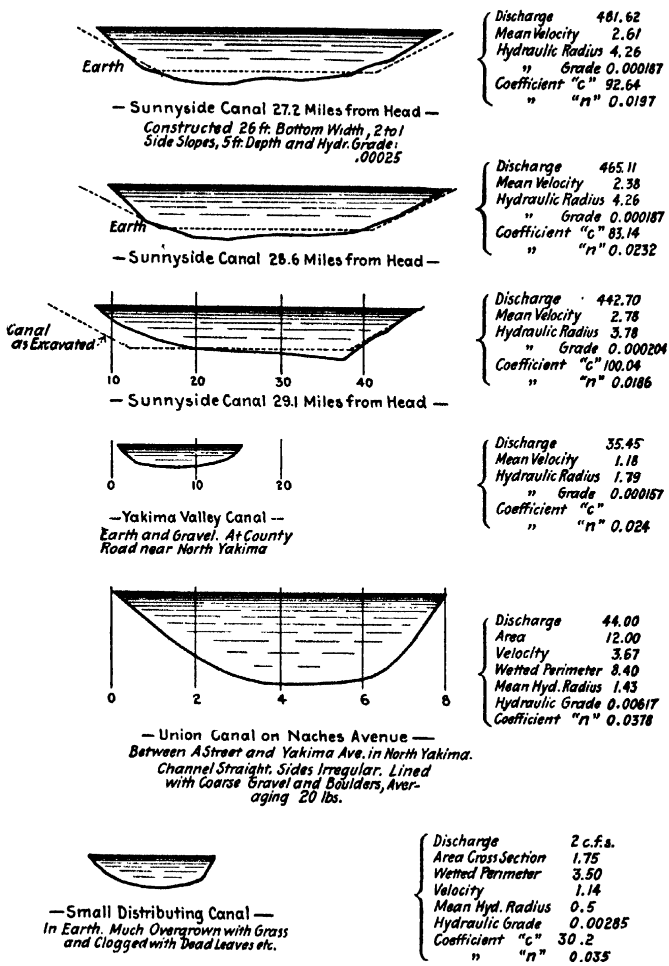


FIG. 21.—Coefficients of roughness of earth canals.

John R. Freeman, Frederic P. Stearns, and James D. Schuyler (*Eng. News*, Jan. 24, 1907) reported to the city of Los Angeles, in connection

with the proposed plans for building an aqueduct to bring the new water supply from the mountains, that—

"We suggest the use of the following coefficients in the Kutter formula:

"For open masonry conduits of cement or smoothly plastered masonry $n = 0.018$.

"For concrete-lined tunnels, or covered masonry conduits, $n = 0.014$.

"For steel pipe with rivet heads and seams projecting on the interior $n = 0.016$.

"For earth canals with bottom as left by dredging, $n = 0.0275$."

Theron A. Noble reported, in a valuable article upon "Economic Canal Location" (Proc. Pacific Northwest Soc. C. E., June, 1907) the values given in Fig. 21 for the coefficient of roughness in canals in earth, as determined in the Sunnyside, Yakima Valley and Union Canals, in connection with the Tieton project of the U. S. Reclamation Service upon the Yakima River in the state of Washington.

The Bureau of Surveys of Philadelphia, Pa. (1909) had a series of observations made, of the values of the coefficient of roughness, n, of certain of the large sewers in that city, with the following result:

	n
Old sewers, brick bottom not clean	0.017
Old sewers, stone block bottom clean	0.017
New sewers, stone block bottom clean	0.016
New sewers, brick bottom clean	0.015
Concrete or brick sewer, vitrified shale brick invert, clean	0.012 to 0.013
Concrete sewers, granolithic finished bottom	0.011
Open channel box, planed planks	0.011
Old sewers, bad or dirty bottoms	0.017 to 0.020

James A. Cushman (1911) in an interesting article upon "Coefficients of Flow in the Wachusett Aqueduct," of the Metropolitan Water Works of Boston, Mass. (*Eng. News*, Aug. 29, 1912) gives a summary of the gagings made there. The section of the conduit, Fig. 22, shown here is horseshoe in shape, lined with brick upon its invert and for a distance of about 5.4 ft. above it, the remainder of the section and roof of the conduit being built of Portland cement concrete. Table 12 gives information available in regard to the ratings and observations which were used to determine the value of n for the conduit below the top of the brick lining.

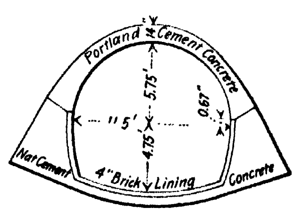


FIG. 22.—Cross section of Wachusett aqueduct.

TABLE 12.—COEFFICIENT OF ROUGHNESS OF WACHUSETT AQUEDUCT

Rating date	Aqueduct last cleaned	Days in use since cleaning	Q flowing c. f. s.	Coefficient of roughness "n"
Apr. 20, 1899......	Apr. 5, 1899 ...	15	178 5	0.01239
Apr 21, 1899. . .	Apr. 5, 1899......	16	145 2	0 01238
May 15, 1899......	Apr. 5, 1899.. ...	40	113 9	0.01234
May 17, 1900......	Apr. 5, 1899.. .	401	89.6	0.01269
May 17, 1900... .	Apr. 5, 1899.	401	92 4	0 01288
May 9, 1901.	Apr. 14, 1901.....	91	85 6	0.01181
Apr. 26, 1907	Dec. 21, 1906	122	220 3	0.0120
Water above top of brick lining				
Mar. 24, 1904.. ...	Dec. 8, 1903......	66	350 5	0.01216
Feb. 13, 1912 . ..	Apr. 15, 1911.	219	492 7	0 01153
Mar. 13, 1912	Apr. 15, 1911	250	428 4	0.01171

With the exception of two ratings in 1900 and one in 1907, the observations were made after cleaning. The investigation showed that as the water rose above the top of the brick lining, the value of n decreased, and the coefficient of flow, c, in the Chezy formula increased.

The values of n derived for use in Kutter's formula were

For the brick-lined surface	0 0123
For the combination of brick and concrete surface.	0.01135

Ganguillet and Kutter give for new well-laid brickwork, 0.011 to 0.012, and for cement (one-third sand), 0.011. It seems reasonable to assume that, had this conduit been built entirely of concrete as smooth as the upper part was built, the value of n could have been taken as low as 0.0112 and perhaps lower, at the point of maximum flow.

The values of c derived and the quantities flowing were as follows:

	c	Q
Aqueduct ¼ full....................	136	61 m.g d.
Aqueduct ½ full	144	156 m.g.d.
Aqueduct ¾ full	154	271 m.g.d.
Aqueduct at maximum capacity	158	363 m.g.d.
Aqueduct full..	157	349 m.g.d.

Cushman further suggests as the formula for determining the value of the coefficient, in the Chezy formula—

$$c = 124.5r^{0.15} \text{ for flows below the brick}$$

lining, and for flows above the brick lining,

$$c = 1.515r^{0.15} \div n$$

These formulas are similar in form to those found for the Sudbury, Stony Brook and Croton Aqueducts, for which the respective values of c were found to be

$$127r^{0.12},\ 122.6r^{0.07},\ \text{and}\ 124r^{0.06}$$

"The Sudbury Aqueduct gagings by Fteley and Stearns developed a value of

$$v = 127r^{0.12}\sqrt{rs} = 127r^{0.62}s^{0.50}$$

In a portion of this conduit where the brick lining was coated with pure cement, the coefficient was found to be from 7 to 8 per cent. greater than $127r^{0.12}$. In another portion where the brick lining was covered with a cement wash laid on with a brush, the coefficient was from 1 to 3 per cent. greater. For a long tunnel in which the rock sides were ragged, but with a smooth cement invert it was found to be about 40 per cent. less. Owing to the fouling of such conduits as the result of vegetable growths and the deposition of materials from the water, a diminution in capacity of from 10 to 20 per cent. with age may be expected, and accordingly corresponding allowances should be made in the design." (Merriman, "Treatise on Hydraulics," p. 301.)

Walter W. Patch, 1902, then Assistant Engineer of the Sudbury Department, Metropolitan Water Works of Boston, in *Eng. News*, June 12, 1902, p. 488, describes his measurements of the flow of water in the Sudbury and Cochituate Aqueducts. "While the methods of metering the flow therein discussed are valuable, the matter of greatest interest here is the discussion upon the rapid loss in carrying capacity of these conduits after cleaning and the determination of a coefficient of cleanliness which could be applied, by means of periodic meter gagings at certain stations upon the aqueduct, to the discharges computed by a formula based upon the flow of the conduit when clean, in order to give the actual flow under existing conditions of cleanliness. These coefficients were found to vary from 89.7 to 103.6 per cent. upon the Sudbury Aqueduct, and from 94.4 to 107 per cent. on the Cochituate Aqueduct." Mr. Patch concluded that "unless the degree of cleanliness of the interior of the aqueduct is known, the computed flows may be 10 per cent. in error." The variation found during the period of one year is shown in Fig. 23.

The sudden increases in the value of the coefficient of cleanliness are due to cleaning of a portion or the whole of the aqueduct.

Similar conditions will be found in sewers due to the formation of scum and growths adhering to the walls, and deposits upon the bottom of the conduit.

Leroy K. Sherman writes (*Eng. News*, July 27, 1911) that upon the rock sections of the Chicago Main Drainage Canal, values of n between 0.024 and 0.029 have been observed, the latter probably being the more accurate. The channel is 160 ft. wide and 23 to 24 ft. deep. The sides are vertical, channeled smooth in limestone rock; bottom rough.

In the experiments Q varied from 6000 to 8000 cu. ft. per second, and S from 0.00001 to 0.00002. This channel may be outside of the limiting range for a constant value of n in the Kutter formula.

TABLE 12.—COEFFICIENT OF ROUGHNESS OF WACHUSETT AQUEDUCT

Rating date	Aqueduct last cleaned	Days in use since cleaning	Q flowing c. f. s.	Coefficient of roughness "n"
Apr. 20, 1899	Apr. 5, 1899	15	178.5	0.01239
Apr. 21, 1899	Apr. 5, 1899	16	145.2	0.01238
May 15, 1899	Apr. 5, 1899	40	113.9	0.01234
May 17, 1900	Apr. 5, 1899	401	89.6	0.01260
May 17, 1900	Apr. 5, 1899	401	92.4	0.01288
May 9, 1901	Apr. 14, 1901	91	85.6	0.01181
Apr. 26, 1907	Dec. 21, 1906	122	220.3	0.0120
Water above top of brick lining				
Mar. 24, 1904	Dec. 8, 1903	66	350.5	0.01216
Feb. 13, 1912	Apr. 15, 1911	219	492.7	0.01153
Mar. 13, 1912	Apr. 15, 1911	250	428.4	0.01171

With the exception of two ratings in 1900 and one in 1907, the observations were made after cleaning. The investigation showed that as the water rose above the top of the brick lining, the value of n decreased, and the coefficient of flow, c, in the Chezy formula increased.

The values of n derived for use in Kutter's formula were

For the brick-lined surface	0.0123
For the combination of brick and concrete surface	0.01135

Ganguillet and Kutter give for new well-laid brickwork, 0.011 to 0.012, and for cement (one-third sand), 0.011. It seems reasonable to assume that, had this conduit been built entirely of concrete as smooth as the upper part was built, the value of n could have been taken as low as 0.0112 and perhaps lower, at the point of maximum flow.

The values of c derived and the quantities flowing were as follows:

	c	Q
Aqueduct ¼ full	136	61 m.g.d.
Aqueduct ½ full	144	156 m.g.d.
Aqueduct ¾ full	154	271 m.g.d.
Aqueduct at maximum capacity	158	363 m.g.d.
Aqueduct full	157	349 m.g.d.

Cushman further suggests as the formula for determining the value of the coefficient, in the Chezy formula—

$c = 124.5r^{0.15}$ for flows below the brick

lining, and for flows above the brick lining,

$$c = 1.515r^{0.15} \div n$$

These formulas are similar in form to those found for the Sudbury, Stony Brook and Croton Aqueducts, for which the respective values of c were found to be

$$127r^{0.12},\ 122.6r^{0.07},\ \text{and}\ 124r^{0.06}$$

such a tunnel. The jarring of the brickwork undoubtedly disturbs the mortar joints, more or less, so that the bond with the brick will be broken. Slight projections and irregularities will be caused by this shaking of the brickwork and, irrespective of any visible disturbance or distortion of the same, resistances to the flow of water seem to have been created.

"The author (Ericson) from his experience with these as well as other similar cases, is of the opinion that for tunnels or sewers of ordinary sizes and velocities of flow, lined with sewer brick laid in cement mortar, if the brick is properly selected and not too warped or uneven, laid in a workmanlike manner and true to line, not disturbed by blasting, and the mortar joints scraped off flush with the brick, a coefficient of roughness n in Kutter's formula of 0.0130 is readily attainable, if the extraordinary resistances to flow, such as bends, enlargements, etc., are eliminated.

"Under certain circumstances, especially if there has been any disturbance whatever of the brickwork on account of blasting, it will be found advisable and profitable to have in addition the entire interior surface of brick conduits washed with neat cement, the strokes of the brush applying the wash to be always longitudinally parallel with the axis of the conduit. By this method a coefficient of roughness, n, considerably smaller than 0.0130 should be obtained in well-constructed tunnels of the sewer-brick-lined class."

C. F. Schultz (1912) reports finding the coefficient $n = 0.0151$, for use in Kutter's formula, in a test of the flow of the East Side Tunnel of the Cleveland Water Works. This tunnel was built of shale brick laid in natural cement mortar. The mortar projecting on the inner surface of the tunnel was roughly scraped after the centers were struck, but no particular pains were taken to make the work any smoother than ordinary sewer brickwork. The tunnel was 26,000 ft. long and of 9 ft. nominal diameter.

H. D. Newell (*Eng. News*, May 1, 1913, p. 904) reports experiments upon 16-, 30- and 46-in. reinforced concrete pipes built by the United States Reclamation Service. These results may be summarized as in Table 13.

TABLE 13.—FRICTIONAL LOSS IN 16-, 30-, AND 46-IN. CONCRETE PIPES (H. D. NEWELL)

Diameter, in	Length, feet	Z, c f s	Velocity, f p.s.	Total friction head, feet	S per 1000	Chezy formula c	Kutter's formula n	Hazen & Williams formula c
46	9800	46 15	4 00	8 53	0.87	138	0 011	140
.. ... .	.... .	45 90	3.98	7 68	0 78	145	0.0106	148
.. .	.	48 16	4 17	10 61	1.08	130	0 0117	130
.. .	..	48.59	4.21	10 03	1.02	135	0.0113	135
30	5130	17 70	3 61	5 48	1.07	140	0 0103	148
....	..	(16.62	3 43	5.40	1.05	132–135	0 0108	
		17.04)					0 0106	142
.....	3658	13.38	2 73	3 13	0.86	118	0.0119	126
16	700	3 74	2 68	2 67	3.8	75	0.0154	82
....	....	4 87	3.49	3.16	4 5	90	0.0134	98
16	909	3.42	2.45	1.71	1 88	98	0.0125	109

F. H. Newell (1912), Director of the U. S. Reclamation Service, in a personal letter to the writers in answer to a question as to whether his department had made experiments upon the different irrigation canals and pipe-lines built by it, wrote:

"While some observations have been taken on the value of *n* by the engineers of the Reclamation Service, none of these have been brought to a final conclusion so that we feel we cannot add with certainty anything to the subject. In this connection I may state that most of the earthen canals in the service have been designed on the assumption of *n* equalling 0.025. Observations on some of the canals seem to indicate that this value is a little high and it seems probable that final results will show that a value of *n* equalling 0.0225 is more nearly correct. These conclusions are based on rather incomplete observations on comparatively new canals operated at partial capacity, as in most cases it has not yet been necessary to operate the larger main canals up to their maximum limit."

SUGGESTED VALUES OF n FOR SEWER DESIGN

In view of the facts cited, the writers suggest as reasonable values for the coefficient of roughness *n* in Kutter's formula in the case of sewer pipes, conduits and channels, *under reasonably good operating conditions*, the following coefficients:

	n
For vitrified pipe sewers	0.015
For concrete sewers of large section and best work laid on slopes giving velocities of 3 ft. per second or more	0.012
For concrete sewers under good ordinary conditions of work	0.013
For brick sewers lined with vitrified or reasonably smooth hard burned brick and laid with great care, with close joints	0.014
For brick sewers under ordinary conditions	0.015
For brick sewers laid on flat grade and rough work	0.017 to 0.020

Although many engineers employ $n=0.013$ for vitrified pipe sewers, the authors favor $n=0.015$ where the grades permit, in view of the possibility of rough pipe and poor pipe-laying, which will increase the frictional resistance. If $n=0.013$ is assumed, great care must be taken in specifying and accepting materials, to make certain that the character of construction required is obtained.

The Kutter formula is most readily used by means of diagrams. For many years these were collections of curves plotted on ordinary cross-section paper. The advantages of logarithmic paper for plotting such formulas gradually became recognized, and in 1901 John H. Gregory prepared the diagrams shown in Figs. 24, 25, 26, 27 and 28, which are part of a series of labor-saving charts devised by him at that time. The

TABLE 14.—VALUES OF c FOR USE IN THE CHEZY FORMULA

$v = c\sqrt{rs}$. (U. S. Reclamation Service)

n	.009	.010	.011	.012	.013	.015	.017	.020	.025	.030	.035	.040	.050	.060
r	Slope s = 0.00005 = 1 in 20,000 = 0.264 ft. per mile													
0.1	78	67	59	52	47	39	33	26	20	16	13	11	8	7
0.2	100	87	77	68	62	51	44	35	26	21	18	15	11	9
0.3	114	99	88	79	71	59	50	41	31	25	21	18	14	11
0.4	124	109	97	88	79	66	57	46	35	28	24	20	15	12
0.6	139	122	109	98	90	76	65	53	41	33	28	24	18	15
0.8	150	133	119	107	98	83	71	59	46	37	31	27	21	17
1.0	158	140	126	114	104	89	77	64	49	40	34	29	23	19
1.5	173	154	139	126	116	99	87	72	57	47	40	34	27	22
2	184	164	148	135	124	107	94	79	62	51	44	38	30	25
3	198	178	161	148	136	118	104	88	71	59	50	44	35	29
[1] 3.28	201	181	164	151	139	121	106	91	72	60	52	46	36	30
4	207	187	170	156	145	126	111	95	77	64	56	49	39	33
6	220	199	182	168	156	137	122	105	85	72	63	56	45	38
10	234	212	195	181	169	149	134	116	96	82	72	64	53	45
20	250	228	211	196	184	165	149	131	110	96	85	77	64	55
50	266	245	228	213	201	181	165	148	127	112	101	93	79	70
100	275	254	237	222	210	190	175	158	137	123	112	104	90	80
r	Slope s = 0.0001 = 1 in 10,000 = 0.528 ft. per mile													
0.1	90	78	68	60	51	44	37	30	22	17	14	12	9	7
0.2	112	98	86	76	69	57	48	39	29	23	19	16	12	10
0.3	125	109	97	87	78	65	56	45	34	27	22	19	14	12
0.4	136	119	106	95	86	72	62	50	38	31	25	22	16	13
0.6	149	131	118	105	96	81	70	57	44	35	30	25	19	16
0.8	158	140	126	114	103	88	76	63	48	39	33	28	22	18
1.0	166	147	132	120	109	93	81	67	52	42	35	31	24	19
1.5	178	159	144	130	120	103	89	75	59	48	41	35	28	23
2	187	168	151	138	127	109	96	81	64	53	45	39	31	25
3	198	178	162	149	137	119	104	89	71	59	51	45	35	29
4	206	186	169	155	143	125	111	94	76	64	55	49	39	32
6	215	195	178	164	152	134	119	102	84	71	61	54	44	37
10	226	205	188	174	162	143	128	111	92	78	69	62	50	42
20	237	216	200	185	173	154	139	122	102	89	79	71	60	52
50	249	227	211	197	185	166	151	134	114	100	91	83	71	63
100	255	234	218	204	191	172	158	140	121	108	98	91	79	70
r	Slope s = 0.0002 = 1 in 5000 = 1.056 ft. per mile													
0.1	99	85	74	65	59	48	41	32	24	18	15	12	9	7
0.2	121	105	93	83	74	61	52	42	31	25	21	17	13	10
0.3	133	116	103	92	83	69	59	48	36	29	24	20	15	12
0.4	143	125	112	100	91	76	65	53	40	32	27	23	17	14
0.6	155	138	122	111	100	85	73	60	46	37	31	26	20	16
0.8	164	145	131	118	107	91	79	65	50	41	34	29	22	18
1.0	170	151	136	123	113	96	83	69	54	44	37	32	24	20
1.5	181	162	146	133	122	105	91	77	60	49	42	36	28	23
2	188	170	154	140	129	111	97	82	64	54	45	40	31	26
3	200	179	163	149	137	119	105	89	72	59	51	45	35	29
4	205	185	168	155	143	125	111	94	76	63	55	48	38	32
6	213	193	176	162	150	132	117	100	82	69	60	53	43	36
10	222	201	185	170	158	140	125	108	89	76	67	60	49	41
20	231	210	194	180	168	149	134	117	98	85	76	68	57	49
50	240	220	203	189	177	158	143	126	108	94	85	78	66	58
100	245	224	208	194	182	163	148	131	113	99	90	83	72	64

[1] Values of c are the same for all slopes when $r = 3.28$ ft.

TABLE 14.—CONTINUED. VALUES OF c FOR USE IN THE CHEZY FORMULA

$$v = c\sqrt{rs}$$

n	.009	.010	.011	.012	.013	.015	.017	.020	.025	.030	.035	.040	.050	.060
r	Slope s = 0.0004 = 1 in 2500 = 2.112 ft. per mile.													
0.1	104	89	78	69	62	50	43	34	25	19	16	13	10	8
0.2	126	110	97	87	78	65	54	44	32	25	21	18	13	10
0.3	138	120	107	96	87	73	62	50	37	30	24	21	16	12
0.4	148	129	115	104	94	79	68	55	42	33	27	23	18	14
0.6	157	140	126	113	103	87	75	62	47	38	31	27	20	16
0.8	166	148	133	121	110	93	81	67	51	42	35	30	23	18
1.0	172	154	138	125	115	98	85	70	55	45	37	32	25	20
1.5	183	164	148	135	124	106	93	78	61	50	42	37	28	23
2	190	170	154	141	130	112	98	83	65	54	45	40	31	26
3	199	179	162	149	138	119	105	89	71	59	51	45	35	29
4	204	184	168	154	142	124	110	94	76	63	55	48	38	32
6	211	191	175	161	149	130	116	99	81	69	60	53	43	36
10	219	199	183	168	157	138	123	107	88	75	66	59	48	41
20	227	207	190	176	164	146	131	115	96	83	73	66	55	48
50	235	215	198	184	173	154	139	123	104	91	82	75	63	56
100	239	219	203	189	177	158	143	127	108	96	87	80	68	61
r	Slope s = 0.001 = 1 in 1000 = 5.28 ft. per mile													
0.1	110	94	83	73	65	54	45	36	27	21	17	14	10	8
0.2	129	113	99	89	81	66	57	45	34	27	22	18	13	11
0.3	141	124	109	98	89	74	63	51	39	30	25	21	16	13
0.4	150	131	117	105	96	80	69	56	43	34	28	24	18	14
0.6	161	142	127	115	104	88	76	63	48	39	32	27	21	17
0.8	169	150	134	122	111	94	82	68	52	42	35	30	23	19
1.0	175	155	139	127	116	99	86	71	56	45	38	33	25	20
1.5	184	165	149	136	124	108	93	78	62	50	43	37	29	24
2	191	171	155	142	130	112	98	83	66	54	46	40	31	26
3	199	179	163	149	138	119	105	89	71	59	51	45	35	29
4	204	184	168	154	142	124	110	93	75	63	54	48	38	32
6	211	190	174	160	149	130	116	99	81	68	59	52	42	36
10	218	197	181	167	155	136	122	105	87	74	65	58	47	40
20	225	205	188	175	163	144	129	113	94	81	72	65	54	47
50	232	212	196	182	170	151	137	120	101	89	79	72	61	54
100	236	216	200	186	174	155	141	124	105	94	85	77	66	59
r	Slope s = 0.01 = 1 in 100 = 52.8 ft. per mile													
0.1	110	95	83	74	66	54	46	36	27	21	17	14	10	8
0.2	130	114	100	90	81	67	57	46	34	27	22	19	14	11
0.3	143	125	111	100	90	76	64	52	39	31	25	22	16	13
0.4	151	133	119	107	98	82	70	57	44	35	29	24	18	14
0.6	162	143	129	116	106	90	77	64	49	39	33	28	21	17
0.8	170	151	135	123	112	95	82	68	53	43	35	31	23	19
1.0	175	156	141	128	117	90	87	72	56	45	38	33	25	20
1.5	185	165	149	136	125	107	94	79	62	51	43	37	29	24
2	191	171	155	142	130	112	99	83	66	55	46	40	31	26
3	199	179	162	149	138	119	105	89	71	59	51	45	35	29
4	204	184	167	154	142	123	109	93	76	63	55	48	38	32
6	210	190	173	160	148	129	115	99	81	68	59	52	42	36
10	217	196	180	166	154	136	121	105	86	74	65	58	47	40
20	225	204	187	173	161	143	128	112	93	80	71	64	53	46
50	231	210	194	181	168	150	135	119	100	87	78	71	60	53
100	235	214	197	184	172	153	139	122	104	91	82	75	65	58

Note.—For slopes greater than 0.01 c remains nearly constant.

logarithmic method of plotting has been used by the authors in plotting Figs. 29, 30 and 31, in which the arrangement adopted results in a more open diagram than that of Mr. Gregory.

Engineers' Opinions Regarding "n".—The opinions regarding the values of n of a number of engineers were sought by the authors and are reported here, not only as affording a valuable statement of current (1913) practice, but also as illustrating the important part which experience must play in the design of a successful sewerage system. In the selection of minimum grades and the estimation of the discharge of sewers, sound judgment is absolutely essential.

John W. Alvord reported that for pipe sewers Alvord & Burdick used a diagram computed for $n=0.013$. For brick sewers they were accustomed to use $n=0.014$, and sometimes, when they desired to be particularly liberal, they used 0.015, it being their idea that new rough brickwork probably produced a coefficient of 0.014, while an old sewer would rise to 0.015 on account of the slime. Concrete sewers were computed with $n=0.015$, but they have grown into the belief after completing a number of miles of concrete sewers that they did not ordinarily secure the work in the district about Chicago which justified much less than $n=0.016$. Mr. Alvord called attention to a paper by John Ericson, on "Investigations of Flow in Brick-lined Conduits," in the *Journal of the Western Society of Engineers*, October, 1911, in which the coefficient for brick tunnels considerably slimed through years of use was found to range from 0.0137 to 0.0167, with an average of 0.01385. Mr. Ericson's opinion was that 0.013 was readily obtainable with reasonably good work.

George G. Earl stated that in New Orleans they used a value of $n=0.011$ for pipe sewers, and from 0.012 to 0.013 for smooth well-built brick or concrete sewers and masonry-lined and covered canals, with smooth concrete V-shaped bottoms, and either brick or concrete sides.

George W. Fuller stated that he assumed n as 0.013 for vitrified pipe and for concrete sewers more than 24 in. in diameter, and 0.015 for brick sewers. These values were higher than were used in some projects which had his endorsement; for instance in that at New Orleans the design was based on $n=0.011$ for vitrified pipe sewers, and $n=0.013$ for brick sewers. For a good many years the Hering & Fuller practice was to use 0.015 for outfall sewers whether of concrete or brick, owing to the custom of preparing specifications so as to take alternate bids for both classes of structures. This was also the case in instances like the report on the Passaic Valley Trunk Sewer in 1908, where the structure was figured to be of concrete, although that material had not been finally passed upon.

John H. Gregory wrote that in working out the main line of the Passaic

Valley concrete sewer he used $n = 0.015$. On branch lines of vitrified pipe he used $n = 0.013$.

T. Chalkley Hatton reports the results of experiments on the flow of water in two 24-in. sewers built with 3-ft. lengths of pipe and with cement joints, at Carlisle, Pa. Experiments on a section 4660 ft. long having a grade of 0.077 per cent., and having bends at five manholes with depths of water of 5 and 12 in., gave $n = 0.0128$ and $n = 0.0112$, respectively. One experiment on another section 2095 ft. long and having one bend at a manhole and a grade of 0.04 per cent., gave with a depth of 12 in. $n = 0.0111$, as computed by the authors from Mr. Hatton's data.

Alexander Potter reported that he was of the opinion that for vitrified pipe and small brick sewers the coefficient of roughness ranged from 0.013 to 0.0145, and the value of 0.014 represented average conditions of roughness and depth of flow found in practice.

This practice is based to a considerable extent upon the results of observations made on the joint trunk sewer system in northeastern New Jersey, where the contributing flows from various municipalities are measured by 13 automatic gages keeping a continuous record of the depth of the discharge. Once a week the charts are taken out and new blanks substituted, and as a check on the readings of each chart the

TABLE 15.—MEASURED AND COMPUTED VELOCITIES AND THEIR PERCENTAGE RATIO. (POTTER)

M.V., measured velocity, feet per second; *C.V.*, computed velocity; *P.R.*, percentage ratio

Ratio of depth of flow to diameter		0 20	0 30	0.40	0.50	0 60	0.70
Gage No. 60......	*M.V.*	......		3 18	3.45	3 65	3 84
42-in. brick sewer..	*C.V.*	......	....	3 08	3.49	3.77	3.98
0.13% grade......	*P.R.*	......	......	103 2	98.9	96 8	96.5
Gage No. 53½......	*M.V.*	.....	2.36	2 70	2.92	3 20	3.36
20-in. pipe sewer..	*C.V.*	......	2 20	2.69	3.05	3.30	3.48
0.28% grade......	*P.R.*	.....	107 2	100.5	95.7	97.0	96 6
Gage No. 4½.......	*M.V.*	.....	....	4 23	4.81	5.20	5 54
22-in. pipe sewer...	*C.V.*	....	....	4 27	4 85	5.24	5.53
0.6% grade.......	*P.R.*	......	....	99 1	99 2	99.2	100.2
Gage No. 35.......	*M.V.*	....	2.18	2 50	2.73	2 90	...
24-in. pipe sewer...	*C.V.*	.....	1.96	2.40	2.72	2 94	......
0.18% grade......	*P.R.*	.....	111 3	104 2	100.4	98 7	.
Gage No. 72	*M.V.*	1.78	2 05	2 58	......	......	..
22-in. pipe sewer	*C.V.*	1 62	2 08	2.55	.....	....	.
0 22% grade	*P.R.*	109 8	99 7	98 1	...	.	.

Percentage ratio of 109 corresponds to $n = 0.013$.
Percentage ratio of 100 corresponds to $n = 0.014$.
Percentage ratio of 92 corresponds to $n = 0.015$.

operator determines the actual depth of flow and, by means of floats, the velocity of the sewage at that point. The average results of 50 to 60

obervations made in 1906 to 1909, inclusive, on sewers built in 1903, are given in Table 15.

Mr. Potter was of the opinion that in a sewer which had been in use for some time the coefficient of roughness was a minimum when the sewer flowed less than three-eighths full. Under such conditions the coefficient was about 0.013, he believed. As the depth of flow became greater than this the coefficient of roughness apparently increased, especially in brick sewers, he stated.

Effect of Variation in Assumed Value of n.—Ernest W. Schoder (*Eng. News*, Aug. 22, 1912) called attention to the fact that the percentage error resulting from a wrong assumption as to the value of the coefficient of roughness n can readily be approximately determined for the Kutter and Bazin formulæ in spite of the apparently complicated nature of their coefficients. Broadly speaking, the following relations hold:

1. The slope s varies as n^2, almost exactly for all values of the hydraulic radius r greater than 1 ft.

2. The velocity v varies inversely as n, exactly for $r=$about 2 ft. and approximately for other values.

Corresponding to these relations we may state that a certain percentage of uncertainty in the value of n produces:

1. Double that percentage of uncertainty in the slope necessary for a fixed discharge.

2. The same percentage of uncertainty, but in opposite direction, in the velocity of discharge resulting from a fixed slope, if the slope is assumed to be greater than 0.0001.

As an illustration of the convenience of this knowledge, suppose that in designing a canal, it is uncertain what value in the range between 0.017 and 0.020 to choose for n. This is an uncertainty of about 8 per cent. either way from the mean value and represents a probable occurrence in practice. We can state at once that the uncertainty in discharge as caused by ignorance concerning n will be about 8 per cent. and in required slope, about 16 per cent.

The diagrams prepared by Schoder are given in Figs. 32 and 33; reference may also be made to diagrams 5 and 15 of Swan and Horton's "Hydraulic Diagrams."

The Limitations of Kutter's Formula.—Being essentially an empirical formula, based upon actual gagings, it is of importance to remember the limits within which observations have been made and further to remember that while velocity varies approximately as the square root of the head under velocities corresponding to the ordinary conditions of flow, it varies more nearly directly as the head under extremely low velocities. Within the ordinary velocity limits of from 1 to 6 ft., the formula finds its best application. It is fairly reliable up to 10 ft. per second velocity.

For special cases, which may be outside of the range of the formula such as 20 ft. per second or higher velocity, the engineer should make reference to the original data, published in Hering and Trautwine's translation of Ganguillet and Kutter's work, and that of other writers upon hydraulics since that time.

Hughes and Safford ("Hydraulics," p. 343) have summed up the application of this formula in an excellent manner as follows:

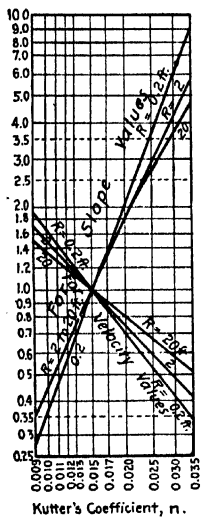


FIG. 32.

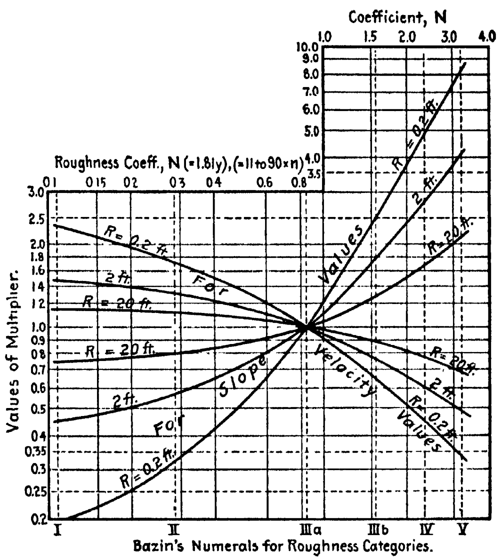


FIG. 33.

FIG. 32.—Relation between Kutter's n and corresponding slopes and velocities.

FIG. 33.—Relation between Bazin's N and corresponding slopes and velocities.

"There is a wide range in the magnitude of the streams on which this formula is based (from hydraulic radii of 0.28 to 74.4 ft.); but a study of the data on which the formula is based, as given in the authors' book, has led to the following conclusions:

That, for hydraulic radii greater than 10 ft., or velocities higher than 10 ft. per second, or slopes flatter than 1 in 10,000, the formula should be used with great caution. For hydraulic radii greater than 20 ft., or velocities higher than 20 ft. per second, but little confidence can be placed in results.

That, considering the variable accuracy of the data on which the formula is based, results should not be expected to be consistently accurate within less than about 5 per cent.

That, for any slope steeper than 0.001 the values of c computed for $s =$ 0.001 may be used with errors less than the probable error in the ordinary use of Kutter's formula.

That between slopes of 0.001 and 0.004 the maximum variation (at the extreme values of n and r) in c is about 4 per cent.; for such values as fall within the range of ordinary practice the maximum variation is but 2 per cent.

That between slopes of 0.0004 and 0.0002 the maximum variation is about 5 per cent., but for such values as fall within the range of ordinary practice the maximum is less than 3 per cent.

That for higher values of s the divergence in the values of c increases; but the occasions when slopes flatter than 0.0004 are to be considered in design are not common, and when they do occur they are usually for structures of such high character that they warrant special study and some basis in addition to a general empirical coefficient. And considering that a degree of precision of 0.001 is rarely exceeded in leveling for ordinary construction work, and that in picking out the value of n, a variation of 0.001 for small values of n and r may change the value of c as much as 17 per cent., and for moderate values as much as 5 to 8 per cent., it should be obvious that hair-splitting calculations with the Kutter formula are a needless waste of time, producing merely mechanical accuracy instead of a high degree of precision."

Effect of Ice.—The effect of an ice-sheet upon a canal, in reducing the flow, is of importance as it increases the area producing frictional resistance to flow. This is indicated clearly by Fig. 17, showing the distribution of velocity in a vertical section of a flowing stream.

For a very interesting "Determination of Stream Flow during the Frozen Season " by H. K. Barrows and Robert E. Horton, reference may be had to Water Supply and Irrigation Paper No. 187 (Series M, General Hydrographic Investigations 19, published in 1907 by the U. S. Geological Survey) in which this subject is fully discussed in the light of a large number of actual observations and records.

HAZEN AND WILLIAMS' FORMULA

Of late years, several exponential formulas for the flow of water in pipes have been developed. Of these the most important is that developed by Allen Hazen and Gardner S. Williams, which agrees closely with observed results and has the great merit that it can be applied with facility through the special slide rule designed and graduated for the solution of problems by it. Tables have also been prepared covering its application. Inasmuch as careful comparison of this formula has been made with the better known Kutter's formula, and as the use of the slide-rule is not only convenient but effects a very considerable saving in time in making many hydraulic computations, this formula is of particular importance. While this formula has had application most often to pipes discharging under pressure, it may also be used in sewer computations.

$$v = cr^{0.63}s^{0.54}.001^{-0.04}$$

in which v = velocity, in feet per second
c = coefficient of roughness
r = hydraulic mean radius
s = slope

The authors say of it,

"The exponents in the formula used were selected as representing as nearly as possible average conditions, as deduced from the best available records of experiments upon the flow of water in such pipes and channels as most frequently occur in water-works practice. The last term, $0.001^{-0.04}$, is a constant, and is introduced simply to equalize the value of c with the value in the Chezy formula, and other exponential formulas which may be used at a slope of 0.001 instead of at a slope of 1." (Hazen & Williams, "Hydraulic Tables," pp. 1 and 2.)

This formula may also be written

$$H_f = 3.02121 \frac{V^{\frac{11}{1111}}}{c^{\frac{11\ 1111}{6}} D^{\frac{7}{6}}} = 3.02121 \frac{V^{1.851851}}{c^{1.851851} D^{1.166666}}.$$

in which H_f = friction head, in feet
V = velocity, in feet per second
c = coefficient of roughness
D = internal diameter of pipe, in feet

With regard to the coefficients to be used in this formula in general design, Hazen and Williams suggest the following values for c:

140 for new cast-iron pipe when very straight and smooth;
130 for new cast-iron pipe under ordinary conditions;
100 for old cast-iron pipe under ordinary conditions; this value to be used for ordinary computations anticipating future conditions;

110 for new riveted steel pipe;
95 for steel pipe under future conditions;

140 for new lead, brass, tin or glass pipe with very smooth surface,
130 to 120 ditto, when old;
120 for smooth wooden pipe or wooden stave pipe;

140 for the masonry conduits of concrete or plaster with very smooth surfaces and when clean;
130 ditto, after a moderate time when slime-covered;
120 ditto, under ordinary conditions;

110 for cement-lined pipe (Metcalf);
100 for brick sewers in good condition;

110 for vitrified pipe sewers in good condition;

The Hazen and Williams formula reduces to the following forms for the given values of c

when $c = 100$, $v = 131.8\ r^{0.63}s^{0.54} = 55.0\ d^{0.63}s^{0.54}$
when $c = 110$, $v = 145.0\ r^{0.63}s^{0.54} = 60.5\ d^{0.63}s^{0.54}$
when $c = 120$, $v = 158.2\ r^{0.63}s^{0.54} = 66.0\ d^{0.63}s^{0.54}$
when $c = 130$, $v = 171.4\ r^{0.63}s^{0.54} = 71.6\ d^{0.63}s^{0.54}$
when $c = 140$, $v = 184.6\ r^{0.63}s^{0.54} = 77.1\ d^{0.63}s^{0.54}$

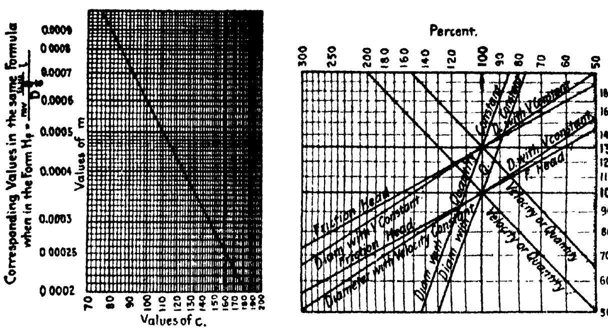


Diagram A.
Flow of Water in Pipes,
Corresponding Values of c and m
in Hazen-Williams Formula

Diagram 1.
Rate of Variation of Certain Factors
in Hazen-Williams Formula.
$V = cR^{0.63}S^{0.54}(0.001)^{-0.04}$

FIG. 34.—Relations between factors in Hazen-Williams formula.

when $c = 99.8-$ $H_f = 0.000600\ v^{\frac{11\,1111}{6}}\ l/D^{\frac{7}{6}}$
$c = 100.$ $H_f = 0.000598\ v^{\frac{11\,1111}{6}}\ l/D^{\frac{7}{6}}$
$c = 104.6-$ $H_f = 0\ 000550\ v^{\frac{11\,1111}{6}}\ l/D^{\frac{7}{6}}$
$c = 110$ $H_f = 0\ 000501\ v^{\frac{11\,1111}{6}}\ l/D^{\frac{7}{6}}$
$c = 110.1+$ $H_f = 0.000500\ v^{\frac{11\,1111}{6}}\ l/D^{\frac{7}{6}}$
$c = 116.6-$ $H_f = 0.000450\ v^{\frac{11\,1111}{6}}\ l/D^{\frac{7}{6}}$
$c = 120.$ $H_f = 0.000426\ v^{\frac{11\,1111}{6}}\ l/D^{\frac{7}{6}}$
$c = 124\ 2$ $H_f = 0.000400\ v^{\frac{11\,1111}{6}}\ l/D^{\frac{7}{6}}$
$c = 130.$ $H_f = 0.000368\ v^{\frac{11\,1111}{6}}\ l/D^{\frac{7}{6}}$
$c = 140.$ $H_f = 0\ 000321\ v^{\frac{11\,1111}{6}}\ l/D^{\frac{7}{6}}$

The ratio between the value of c and of m in the Hazen-Williams

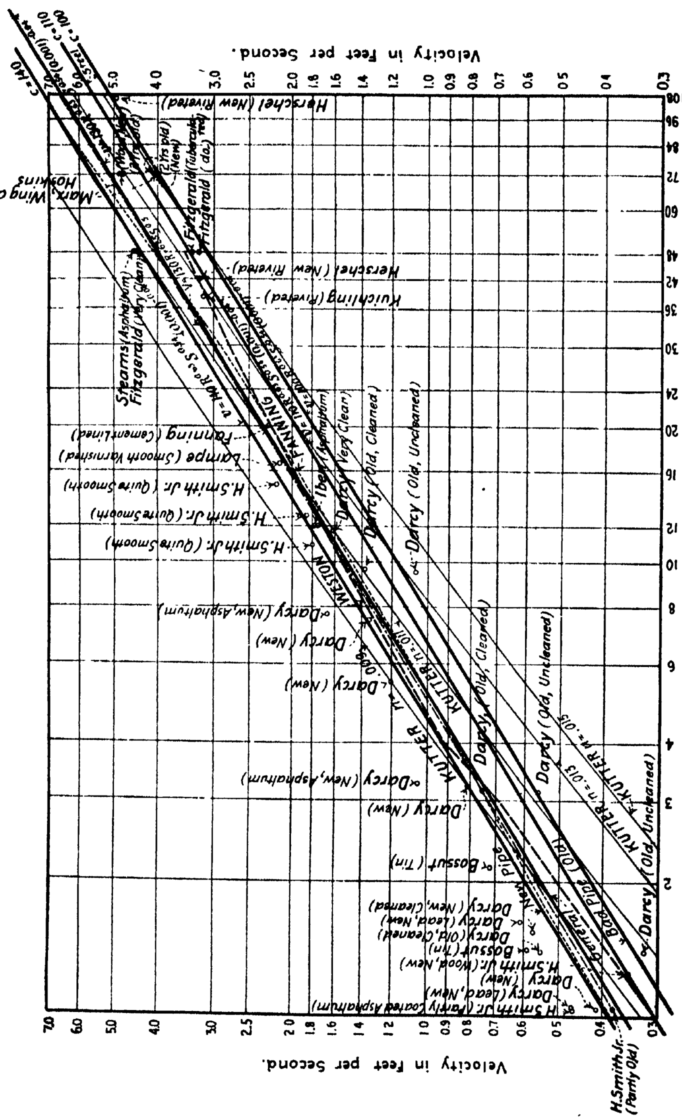


FIG. 37.—Comparison of several formulas for velocity of flow in pipes.

formula is shown in Diagrams A and I, Fig. 34. Figs. 35 and 36 have been plotted for c equal to 100 and 130 respectively and Fig. 37 gives a comparison of the Hazen and Williams formula with others and with experimental results.

The relation between the value of c in the Hazen and Williams formula and the c of the Chezy formula may be found by equating the value of s in these two formulas, which gives the equation

$$(\text{Chezy}) = 1.1506\ c^{0.9259} v^{0.0741} / D^{0.0833}$$

CHAPTER V

QUANTITY OF SEWAGE

Much information relating to the quantity of sewage likely to be, and actually being, produced by municipalities has been published in various papers and reports. As this quantity is a fundamental factor to be considered in the design of sewers, intercepters, pumping stations and treatment works, an effort has been made herein to bring together some of the more significant data and to set forth some of the conditions influencing the volume of sewage.

The quantity of sewage which must be provided for may be considered as made up of definite portions of,

First, domestic and manufacturing sewage, derived primarily from the public water supply carrying the waste products due to modern domestic and industrial conditions;

Second, manufacturing wastes not originating from public water supply, consisting of certain quantities of water procured from other sources such as wells, rivers and lakes, which will be defiled by the processes in which they are used;

Third, the water which finds its way into the sewers through infiltration and which is either ground water, as ordinarily considered or (in close proximity to rivers) may be water filtering through the ground from rivers, and

Fourth, rainfall immediately collected and called "storm water;" this is treated in Chapters VI, VII, VIII and IX.

As it is desirable in designing sewers to provide for the future, estimates of population become necessary in order to ascertain the total amount of sewage of the first three classes for which the sewers must be proportioned.

POPULATION

It is impossible to forecast precisely the population of the city at any definite time in the future or the rate at which the city will grow. However, a consideration of the growth of a city in the past, its location and natural advantages, together with a study of the past growth of other cities now of greater size, makes it possible to prepare a logical estimate of the probable future rate of growth.

The present population, if no recent census has been taken, may be estimated in a number of ways. The most obvious method is to

assume that the rate of growth has been uniform and the same as that between the two most recent census enumerations. Where the number of "assessed polls" is known, it is possible to obtain a fair approximation of the total population by multiplying this figure by a factor obtained by comparing the number of "assessed polls" with the population in past census years. Other factors of similar character may be obtained by use of "school census" returns, the number of voters at recent elections, the number of names in the Directory, or Post Office or Police Department counts. None of these methods is, however, of great value in itself but may be utilized to confirm, or aid in forming, an estimate.

The future population may be predicted in a variety of ways which are

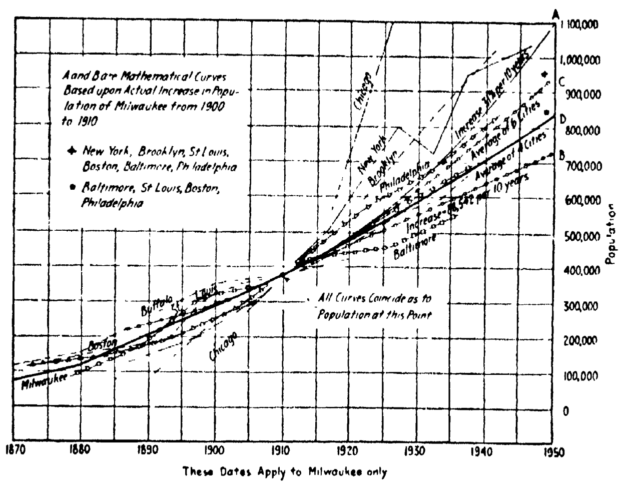


FIG. 50.—Growth of large American cities.

more or less logical, and if employed with care and the data used in applying them are correct, the results will probably average as close to the truth as it is reasonable to expect such prophecies to be. The degree of accuracy is sufficient to enable a sewerage system to be designed with capacity enough to meet the requirements during the term of years for which it is planned, and yet not be of such great capacity that it throws an unwarranted financial burden on the community. These methods of predicting changes in population are:

1. By assuming that the rate of growth between recent census enumerations will remain constant for a considerable number of years.

2. By assuming that the rate of growth can be shown graphically by plotting a curve through the points representing the population of the city at different dates and then extending this curve into future years.

3. By assuming that the rate of growth will show a uniform arithmetical increase from one census year to another.

4. By assuming a steady decrease in the percentage rate of increase as the city grows larger and older.

Assumption of Uniform Rate of Growth.—A prediction of the increase in population, based on the assumption that the rate of growth between recent census years will remain uniform for a considerable future period, is shown by line A in Fig. 50. This undoubtedly gives in many cases, particularly where the communities are young and thriving, results which are too large, as indicated by the records of urban development. In view of this fact, the approval of the method contained in some of the early treatises on sewerage is an indication of the slight basis of fact on which the plans made then rested. For example in Baldwin Latham's "Sanitary Engineering," edition of 1878, the following advice is given:

> "The mode usually adopted in approximating the future population, is to ascertain what has been the prospective rate of increase for a number of years back, and by making the same, or, in some cases, a greater allowance for increase in the future, so to calculate what is likely to be the probable population in years to come. In some districts this mode of estimating the population has been shown to be liable to error, as there are districts, such as manufacturing or suburban districts, located near large centers of population, which are liable to rapid rates of increase, and in some cases the population of particular manufacturing and mining districts has been found to decline."

This method was a favorite one in Germany down to about 1890, when it was discovered that many of the large cities which had increased uniformly from year to year from 1870 to about 1887 or 1888, had suddenly begun to grow at a much more rapid rate. Munich, Leipzig and Cologne showed this change in an astonishing way. Until this rejuvenation took place, it was customary to predict the growth of German cities by the formula, $P = p\,[1 + (f/100)]^n$ where P is the population after n years have elapsed, p is the present population and f is the annual percentage of increase in the population which has been observed. Practically, the growth of many of these cities could be satisfactorily represented by straight lines down to 1887. The growth of the population of the London metropolitan district from 1841 to 1891 was about 20 per cent. every decade, whereas the decennial rate of growth in Berlin and its suburbs has been more rapid and, as is to be expected in a place of such rapid development in population, industries and commerce, the rate of increase has been erratic, like that of many thriving American cities. The method of estimating population by a uniform rate of increase is apparently most

reliable in the case of large and old cities not subject to periods of great commercial or industrial activity.

Graphical Method of Estimating Future Population.—The information furnished by diagrams of the past growth of cities is very instructive, but an attempt to predict the future growth of a city from its past development, by extending the curve of that development, is likely to give misleading results, as will be shown later. Diagrams have a useful place in the study of changes in population, but they are not a substitute for an investigation of the various influences which have affected the city's growth in the past and may affect it in the future.

Arithmetical Increase in Population.—The method of predicting future population which is carried out by assuming that the increase from decade to decade is an arithmetical rather than geometrical progression gives the straight line shown in Fig. 50, line B. An instance of the use of this method was in the preparation of the estimate of the population of the Borough of Manhattan made by the Board of Water Supply of New York City. According to this assumption, the arithmetical increase will become nil when the population reaches 3,000,000, the entire subsequent growth of the city taking place in the other boroughs. Dr. Walter Laidlaw estimated in 1908 that New York's population would increase in an arithmetical rather than geometrical progression, basing this conclusion on the relative growth of New York and the whole country, the probable distribution of future immigrants, and an increasing westward trend of the country's inhabitants.

Decrease in Percentage Rate of Growth as Cities Increase in Size.—As a general rule it is found that the larger the city becomes, the smaller will be the percentage rate of growth from year to year. From the tabulation of the rates of growth of six of the large cities of the country, Table 36, it is apparent that this reduction in the percentage of growth is material.

TABLE 36.—AVERAGE RATE OF GROWTH OF CITIES, AT VARIOUS STAGES OF GROWTH

Size of city	Percentage rate of growth per 10 years						
	Philadelphia	St Louis	Boston	Baltimore	Cincinnati	Milwaukee	Average
100,000–200,000	39.6	103.1	45.5	53.3	52.1	70.0	60.6
200,000–300,000	44.6	76.9	44.6	28.1	21.0	41.0	42.7
300,000–400,000	51.3	21.5	33.3	28.7	....	[1]31.1	33.2
400,000–500,000	39.7	26.9	24.5	20.3	......	......	27.8
500,000–600,000	27.4	24.1	14.9	...	......	.....	22.1
600,000–700,000	20.0	[1]18.9	[1]17.6	....	......	....	18.8
700,000–800,000	24.7	......	.....	...	.....	....	[1]17.7
800,000–900,000	23.2	...	.	...	....	...	[1]17.0

[1] Estimated. *Note*: Figures based on assumed uniform growth between census periods.

TABLE 37.—RATE OF GROWTH OF CITIES FROM DECADE TO DECADE

Decade	Cities between 100,000 and 200,000		Cities between 200,000 and 400,000	
	Number	Rate of increase	Number	Rate of increase
		Per cent.		Per cent.
1840–1850	2	39.5	.....	....
1850–1860	4	33.6	.....	....
1860–1870	6	63.2	.....	....
1870–1880	6	34.8	5	29.6
1880–1890	10	48.7	6	24.0
1890–1900	11	28.7	12	26.3
1900–1910	13	31.5	12	20.3

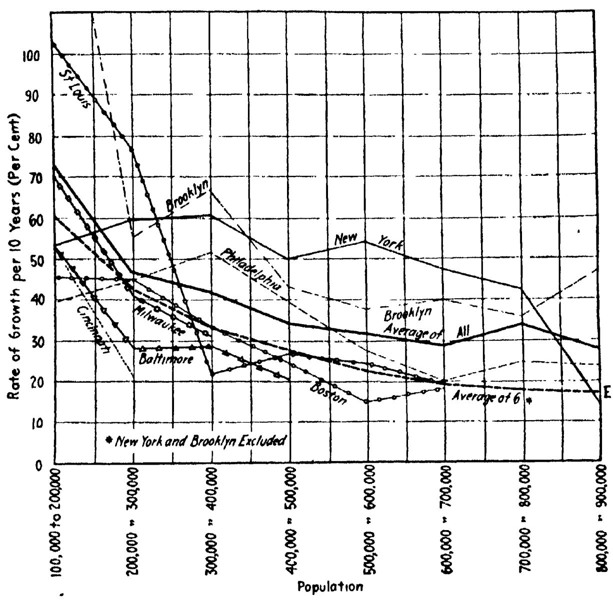


FIG. 51.—Relation of rate of growth to population.

Upon Fig. 51 have been plotted the rates of growth of several of the large cities of the country, showing the ratio of growth to population when they had a population of from 100,000 to 200,000; 200,000 to 300,000, etc. The marked tendency toward a reduced percentage rate of increase is clearly shown in all cases. The heavy solid line shows the average ten-

dency of all and the heavy dotted line the average tendency of all except New York and Brooklyn.

TABLE 38.—RATE OF GROWTH OF AMERICAN CITIES IN 1900-1910

Division	Cities over 100,000 or more in 1910				Cities of 25,000 to 100,000 in 1910			
	No	Aggregate population		Increase, per cent	No	Aggregate population		Increase, per cent.
		1910	1900			1910	1900	
New England	8	1,606,984	1,325,651	21.2	34	1,637,987	1,269,941	29.0
Middle Atlantic	11	8,599,877	6,575,912	30.8	44	2,110,782	1,574,958	34.0
East North Central	10	4,761,966	3,600,614	32.3	38	1,553,809	1,127,923	37.8
West North Central	5	1,575,658	1,208,321	30.4	17	801,931	640,520	25.2
South Atlantic..	4	1,172,021	974,643	20.3	16	712,387	516,427	37.9
East South Central	4	598,082	444,444	34.6	7	289,285	237,257	21.9
West South Central	1	339,075	287,104	18.1	12	636,814	331,409	92.2
Mountain........	1	213,381	140,472	51.9	5	230,995	149,556	51.5
Pacific..	6	1,435,094	727,428	97.3	6	267,688	128,527	108.3
United States	50	20,302,138	15,284,589	32.8	179	8,241,678	5,976,518	37.9
	Cities of 2500 to 25,000 in 1910				Territory rural in 1910			
New England ..	320	2,210,374	1,893,939	16.7	..	1,097,336	1,102,486	-0.5
Middle Atlantic	444	3,012,714	2,156,847	39.7	...	5,592,519	5,146,961	8.7
East North Central	474	3,301,496	2,619,474	26.0	...	8,633,350	8,637,570	.....
West North Central	260	1,496,127	1,173,823	27.5	...	7,764,205	7,324,759	6.0
South Atlantic	190	1,207,745	846,647	42.7	..	9,102,742	8,105,763	12.3
East South Central	115	686,862	504,589	36.1	...	6,835,672	6,361,352	7.5
West South Central	177	981,567	543,223	80.7	..	6,827,078	5,370,669	27.1
Mountain.......	91	503,135	285,304	76.4	..	1,686,006	1,099,325	53.4
Pacific	103	679,547	324,692	109.3	.	1,809,975	1,236,045	46.4
United States	2,173	14,079,567	10,318,538	36.1	.	49,348,883	44,384,930	11.2

The growth of Chicago has been so exceptional that it has not been included in Fig. 51 and it seems probable that the growth of New York and Brooklyn has also been so abnormal that it is hardly safe to base general conclusions on averages in which they are included. The result of this study is illustrated by curve E, Fig. 51, which shows, based upon past experience, the average rate of increase in population which may be expected as the cities increase in size. These results also appear in Table 36; the rate of growth shows a gradual reduction from 60.6 per cent., for cities growing between populations of 100,000 and 200,000, to 17 per cent. for cities growing between populations of 800,000 and 900,000.

An instructive table showing the variation in the rate of increase in cities of different sizes in different parts of the country, between 1900 and 1910, has been prepared by the U. S. Bureau of the Census and is reproduced as Table 38. It shows clearly that local influences are

of great importance in determining the rate of increase of American cities.

Decrease in Percentage Rate of Growth with Age.—In addition to the tendency toward the reduced rate of growth as cities grow larger, there is also a marked tendency toward a decreased growth as the nation grows older. In other words, the rate of growth, as a rule, for cities of similar size, was less between 1900 and 1910 than between 1870 and 1880, as shown by Table 37. This is also true of the population of the entire country, especially during the last half century, as shown by Table 39. Making the corrections suggested by the Census Bureau for the population of 1870, it appears that the rate of growth of the country has decreased from about 35 to 21 per cent. in 100 years, although the actual growth in numbers during this time has increased from decade to decade.

TABLE 39.—POPULATION AND RATE OF GROWTH OF UNITED STATES

Date	Population	Growth during decade	
		Numerical	Per cent
1790	3,929,214	....	......
1800	5,308,483	1,379,269	35.1
1810	7,239,881	1,931,398	36.4
1820	9,638,453	2,398,572	33.1
1830	12,866,020	3,227,567	33.5
1840	17,069,453	4,203,433	32.7
1850	23,191,876	6,122,423	35.9
1860	31,443,321	8,251,445	35.6
1870	39,818,449*	8,375,128	26.6*
1880	50,155,783	10,337,334	26.0
1890	62,947,714	12,791,931	24.9
1900	75,994,575	13,046,861	20.7
1910	91,972,266	15,977,691	21.0

* Census reports claim a deficiency in enumeration of Southern states for 1870. The Census Bureau gives estimated population and percentage as starred. The actual population as returned for 1870 was 38,558,371.

Probably the best result to be derived mathematically may be obtained by assuming, in the light of the statements previously given, a decreasing rate of growth as time goes on, taking into consideration the size of the city at the end of each decade. Such an estimate is shown in Fig. 52. One of the most frequent and useful methods is to base the prediction on the experience of other cities which have already reached and passed the present population of the city under consideration. This is done, as shown in Fig. 50, by arranging the lines indicating the change in population of different cities so that when they have reached the present population of the city under consideration, they all pass through the same point. In this way their behavior after passing this

population may be directly compared. This method may give results somewhat too high as comparison is made with the past growth of cities and, as already pointed out, there is a tendency as time goes on for the rate of increase to become somewhat smaller.

It is usually desirable in such studies to investigate the growth of other cities in the vicinity at the same time as the growth of the city under special consideration, for the information thus obtained will generally reveal any local peculiarities in the increase of population. For instance, in an investigation of the sewerage problems of Fort Wayne, the authors derived assistance from a study of the growth of Indianapolis, Evansville, Terre Haute and South Bend, as well as Fort Wayne. In the case

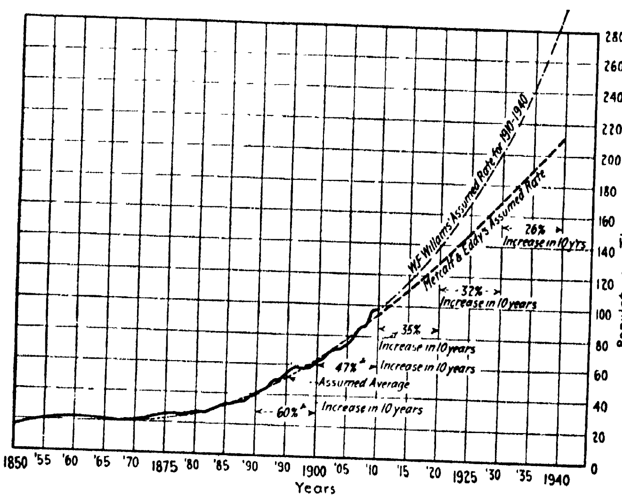


FIG. 52.—Growth of population of New Bedford.

of Fitchburg, Mass., helpful information was obtained from a study of the growth of Salem, Chelsea, Taunton, Haverhill, Newton, Brockton, Malden, Pittsfield, Quincy and Everett.

Increase in Area.—In estimating the probable quantity of sewage to be provided for by intercepting sewers, it is important to take into account the probable increase in the area served by sewers and, in many cases, the probable increase in area within city limits. Such enlargements of area may cause large and sudden increases in population, which, if not anticipated, may cause the overtaxing of interceptors during the period for which they were intended to be adequate. Furthermore, such increases in area require long extensions in main sewers and may result

in greatly increased quantities of ground water made tributary to the intercepters. Where the community is served by combined sewers, there is also the probability that for considerable periods in the future, or until the population becomes quite dense, brooks will be turned into the trunk sewers, thus adding materially to the nominal dry-weather flow of sewage. It is also of vital importance to consider where the estimated increase in population will occur in order that the lower sections of the intercepter may be placed at elevations from which it will be possible to make extensions into new territory that may become populated within the period for which the intercepter is designed.

An interesting illustration of increase in area by annexation is furnished by the growth of Cincinnati in recent years, shown in Table 40, complied from data published in a general report on the disposal of the sewage of Cincinnati submitted in 1913 by H. M. Waite, H. S. Morse and Harrison P. Eddy.

TABLE 40.—ANNEXATIONS TO THE CITY OF CINCINNATI, 1819–1913

Date of annexation	Area annexed (square miles)	Total area (square miles)	Date of annexation	Area annexed (square miles)	Total area (square miles)
1819[1]	....	3.00	1903	5.13	41.96
1849	2.93	5.93	1904	0 47	42.43
1850	0.23	6 16	1905	0.59	43 02
1855	0.77	6.93	1907	0 48	43.50
1870	12 12	19.05	1909	6.03	49.53
1873	4 48	23.53	1910	0 73	50 26
1889	0.20	23.73	1911	16 03	66.29
1896	11.38	35.11	1912	2 45	68 74
1898	0.16	35 27	1913	1.11	69 85
1902	1.56	36 83			

[1] Original city of Cincinnati; incorporated as a town in 1802, as a city in 1819

There is a marked tendency at present, doubtless encouraged by constantly improving transportation facilities, for the inhabitants of cities to move into suburban districts. This condition tends toward a lower density of population, although it is more effective in reducing the probable increase in density than in diminishing existing density. As the suburban areas become more thickly populated, the improvements of the cities are desired there and are ultimately demanded. To secure these, it often becomes necessary for suburban districts to be annexed to the city, thus extending the city limits. It is reasonable, therefore, to expect a city to increase in area as well as population. In making studies of the future sewerage needs of Fort Wayne, for instance, the authors estimated that the area would grow from 8.6 square miles in 1910 to 17.3 square miles in 1950. In a number of places, municipal boundaries have been ignored in water supply and sewerage

undertakings, as at Boston, Mass., and several sections of the territory about New York.

This tendency of large cities to develop by the absorption of adjoining communities, or by the delegation of full authority over certain classes of public works to commissions acting for the entire district served, has led the Bureau of the Census to pay special attention to municipal districts, because "in some cases the municipal boundaries give only an inadequate idea of the population grouped about one urban center; in the case of many cities there are suburban districts with a dense population outside the city limits, which, in a certain sense, are as truly a part of the city as the districts which are under the municipal government." The 1910 census showed that in 25 such metropolitan districts, the average percentage of increase in the cities during the last decade had been 33.2 per cent. and in the suburbs 43 per cent. But these average figures are extremely misleading when used as a guide to the development of the smaller metropolitan districts, because they are greatly influenced by the growth of districts with more than 500,000 population, and the location and age of a city are of much influence on the development of its suburbs as well as of itself. For example, Providence and Detroit had about the same population in 1900, but the development of the Providence metropolitan district in the following decade was only 29.4 per cent. while that of the Detroit district was 57.1 per cent. Furthermore, the development of the Providence suburbs was more rapid than that of the city, whereas the development of Detroit was almost wholly in the city proper.

Density of Population.—The average density of population varies greatly in different cities, as is shown in Table 41. In designing sewers for a community it becomes necessary to estimate the probable distribution of population within the city. This is largely a matter of conjecture, except in the sections of greatest age, as the density may vary from 2 or even less per acre in outlying districts to 150 or more per acre in the most densely settled parts of some large cities. The New York Metropolitan Sewerage Commission estimates that the future density of population in the part of Manhattan which drains into the Hudson River from the Battery to the Harlem River will be 306 persons per acre; that of the part of the Borough of the Bronx draining into the Harlem River will be 239 persons per acre, and that of the district draining into the Lower East River will be 198 persons per acre. The probable lowest density in any district, 8 per acre, will be in the territory draining into the Upper East River. These figures were obtained by taking the probable population of Manhattan and Brooklyn as of 1960; Queens as of 1950 and the Bronx as of 1940. Furthermore, the character of the various parts of a city changes. A residential section of the present decade may become the commercial or manufacturing district of the next decade, or the change may be in the

TABLE 41.—STATISTICS OF THE 50 U. S. CITIES OF OVER 50,000 POPULATION, HAVING THE GREATEST DENSITY OF POPULATION. (COMPILED FROM FINANCIAL STATISTICS OF CITIES, 1910, BUREAU OF THE CENSUS.)

City	Population 1910	Density, persons per acre 1910	Density, persons per acre 1900	Area land surface within city limits, acres, 1910
Hoboken, N. J.	70,324	85	71	830.0
Jersey City, N. J.	267,779	32	25	8,320.0
Somerville, Mass.	77,236	30	40	2,600.0
Baltimore, Md.	558,485	29	26	19,290.0
Boston, Mass.	670,585	27	23	24,743.0
New York, N. Y.	4,766,883	26	19	183,555.0
Passaic, N. J.	54,773	26	13	2,069.2
Cambridge, Mass.	104,839	26	23	4,014.3
Milwaukee, Wis.	373,857	25	22	14,585.8
Altoona, Pa.	52,127	25	23	2,114.6
Paterson, N. J.	125,600	24	20	5,157.0
Reading, Pa.	96,071	24	20	3,965.0
Charleston, S. C.	58,833	24	23	2,406.4
Newark, N. J.	347,469	23	21	14,826.0
Trenton, N. J.	96,815	22	16	4,490.0
Wilmington, Del.	87,411	22	19	4,026.0
Bayonne, N. J.	55,545	22	13	2,577.0
Camden, N. J.	94,538	21	17	4,474.5
Wilkes-Barre, Pa.	67,105	21	16	3,233.0
Lawrence, Mass.	85,892	21	15	4,185.0
Pittsburgh, Pa.	533,905	20	13	26,510.7
Richmond, Va.	127,628	20	28	6,388.0
Johnstown, Pa.	55,482	20	15	2,723.7
Cleveland, Ohio.	560,663	19	17	29,208.8
Philadelphia, Pa	1,549,008	19	16	83,340 0
Chicago, Ill.	2,185,283	19	14	117,793.1
Harrisburg, Pa.	64,186	19	17	3,402.8
Providence, R. I.	224,326	19	15	11,352.2
Norfolk, Va.	67,452	19	16	3,576.1
Detroit, Mich.	465,766	18	16	26,102.6
Allentown, Pa.	51,913	18	21	2,856.4
St. Louis, Mo.	687,029	17	15	39,276.8
Buffalo, N. Y.	423,715	17	14	24,791.0
Covington, Ky.	53,270	17	24	3,083.0
Louisville, Ky.	223,928	17	16	13,229.7
Rochester, N. Y.	218,149	17	14	12,876.3
Evansville, Ind.	69,647	16	14	4,460.0
Savannah, Ga.	65,064	16	18	4,053.0
Schenectady, N. Y.	72,826	15	11	5,000.0
San Francisco, Cal.	416,912	14	12	29,760.0
Columbus, Ohio.	181,511	14	12	13,017.8

TABLE 41.—STATISTICS OF THE 50 U. S. CITIES OF OVER 50,000 POPULATION, HAVING THE GREATEST DENSITY OF POPULATION. (COMPILED FROM FINANCIAL STATISTICS OF CITIES, 1910, BUREAU OF THE CENSUS.) (*Continued.*)

City	Population 1910	Density, persons per acre 1910	Density, persons per acre 1900	Area land surface within city limits, acres, 1910
Albany, N. Y.	100,253	14	14	6,914.0
Bridgeport, Conn.	102,054	13	9	7,906.0
Lowell, Mass.	106,294	13	13	8,308.0
Lynn, Mass.	89,336	13	10	6,943.0
Terre Haute, Ind.	58,157	12	11	7,828.0
Syracuse, N. Y.	137,249	12	10	11,083.6
New Haven, Conn.	133,605	12	9	11,460.0
Dayton, Ohio	116,577	12	13	10,061.0
Youngstown, Ohio	79,066	12	7	6,606.8

Note—Some cities show an apparent decrease in density since 1900, due to the annexation of large areas of adjacent territory.

character of the population from the section containing the homes of people of considerable means to a congested tenement district. These influences may result in increasing the density, causing it to remain nearly stationary, or even decreasing it in some cases.

A study of the growth of different wards in Boston during 15 years reveals some facts which may aid in predicting the growth of other cities of similar character. The statistics are given in Figs. 53 and 54 and in Table 42. The city may be divided for this purpose into outlying sparsely settled regions, good residential districts, fairly densely populated business and commercial districts, and cheap tenement districts. The increase in density of the sparsely settled districts, wards 23, 24 and 25, was very slow, amounting to only 1 or 2 persons per acre per 10 years. When, however, such districts became fairly well built-up and desirable residential sections, with densities of about 20 to 25 per acre, the increase became rapid, amounting for example in wards 20 to 23, to from 5 to 13 persons per acre per 10 years. The lodging house districts and business sections of those in a transitory stage remained nearly uniform or even decreased in density under certain conditions. The sections with the tenement houses of lowest rental, as Ward 8, appear to be increasing rapidly in spite of a density already very great. In fact, the greatest increase in the whole city in the past 15 years has taken place in those sections, and it appears to be very hazardous to assume that the density in such districts, because it is already high, will not go on increasing. Where a district is in a transitory stage, as between a place for business and a place for residence, its ultimate course may largely affect the density. If it becomes commercial the density may not change greatly or may decrease,

TABLE 41.—STATISTICS OF THE 50 U. S. CITIES OF OVER 50,000 POPULATION, HAVING THE GREATEST DENSITY OF POPULATION. (COMPILED FROM FINANCIAL STATISTICS OF CITIES, 1910, BUREAU OF THE CENSUS.)

City	Population 1910	Density, persons per acre		Area land surface within city limits, acres, 1910
		1910	1900	
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Newark, N. J.	347,469	23	21	14,826.0
Trenton, N. J.	96,815	22	16	4,490.0
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Cleveland, Ohio.	560,663	19	17	29,208.8
Philadelphia, Pa.	1,549,008	19	16	83,340.0
Chicago, Ill.	2,185,283	19	14	117,793.1
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Allentown, Pa.	51,913	18	21	2,856.4
St. Louis, Mo.	687,029	17	15	39,276.8
Buffalo, N. Y.	423,715	17	14	24,791.0
Covington, Ky.	53,270	17	24	3,083.0
Louisville, Ky.	223,928	17	16	13,229.7
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Savannah, Ga.	65,064	16	18	4,053.0
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San Francisco, Cal.	416,912	14	12	29,760.0
Columbus, Ohio	181,511	14	12	13,017.8

whereas if it changes into a cheap tenement region, the density may go on increasing to a very high figure.

TABLE 42.—GROWTH IN POPULATION OF THE WARDS OF THE CITY OF BOSTON

Ward	Area land in acres	1895		1900		1905		1910	
		Popul per acre	Per cent of popul	Popul. per acre	Per cent. of popul.	Popul per acre	Per cent. of popul	Popul. per acre	Per cent. of popul
1	1,188	17.7	4.23	19.2	4.07	21.4	4.27	24.9	4.43
2	357	60.5	4.34	64.2	4.09	72.6	4.35	80.7	4.30
3	332	42.0	2.81	43.9	2.60	44.7	2.49	46.2	2.29
4	301	44.4	2.69	44.0	2.36	41.5	2.10	44.1	1.98
5	207	62.7	2.61	62.0	2.29	61.7	2.12	61.9	1.91
6	293	95.1	5.61	104.3	5.45	102.3	5.04	122.0	5.33
7	394	43.0	3.42	37.5	2.64	39.5	2.62	37.9	2.22
8	171	135.0	4.65	168.5	5.14	180.4	5.17	190.0	4.84
9	186	124.5	4.66	132.0	4.38	118.9	3.72	141.5	3.94
10	394	57.2	4.54	56.2	3.95	60.5	4.00	64.3	3.78
11	663	30.0	4.01	29.1	3.44	33.7	3.75	41.4	4.09
12	235	92.0	4.35	100.6	4.21	92.5	3.65	103.4	3.62
13	611	40.7	5.01	37.4	4.07	35.4	3.64	35.3	3.22
14	405	47.4	3.86	53.0	3.82	54.7	3.72	58.2	3.52
15	277	67.2	3.75	71.1	3.51	73.3	3.41	76.6	3.16
16	564	28.9	3.28	35.5	3.57	38.9	3.68	45.6	3.82
17	460	45.8	4.25	54.4	4.46	52.8	4.08	57.4	3.94
18	220	98.6	4.36	101.9	3.99	100.6	3.72	103.3	3.39
19	760	29.1	4.50	35.7	4.85	38.4	4.91	41.7	4.73
20	1,716	12.6	4.33	19.0	5.80	24.4	7.02	32.5	8.31
21	640	30.1	3.88	37.3	4.26	41.5	4.46	50.5	4.55
22	760	29.2	4.49	33.7	4.57	36.5	4.66	38.1	4.47
23	7,617	2.4	3.68	3.1	4.21	3.5	4.44	4.0	4.57
24	3,252	5.6	3.67	8.3	4.83	9.7	5.32	11.6	5.63
25	2,740	5.5	3.02	7.0	3.44	8.0	3.66	9.7	3.96
Av. density		20.0		22.7		24.1		27.1	

Similar tendencies in Chicago are indicated in a report on sewage disposal in the Chicago Sanitary District, by G. M. Wisner. The average density of the four most densely populated wards increased from 76.2 to 86.2 in 10 years, Table 43. The tendency for the density of business sections to either stand still or decrease somewhat is shown in Fig. 55, from Mr. Wisner's report. It should be borne in mind, however, that this refers to resident population and that the number of people inhabiting the district during the business hours is probably increasing at a rapid rate.

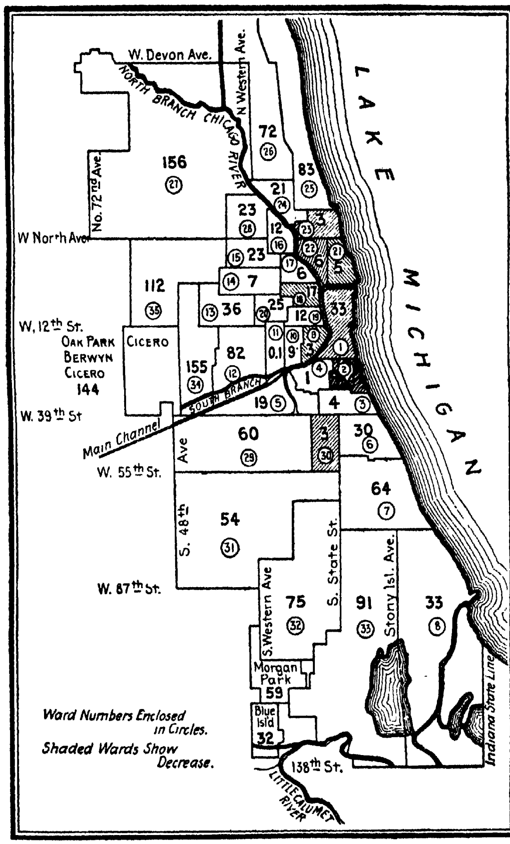


Fig. 55.—Percentage of change in population in the wards of Chicago.

TABLE 43.—POPULATION OF CHICAGO BY WARDS

Ward	Area acres[1]	Population		Per cent. change	Density	
		1900	1910		1900	1910
1	1,440	43,764	29,528	− 33 0	30.4	20.5
2	800	44,583	42,801	− 4.0	55.6	53.5
3	960	44,425	46,135	4.0	46.3	48.1
4	960	49,058	49,650	1.0	51.1	51.7
5	2,240	48,206	57,131	19.0	21.5	25.5
6	1,600	57,831	75,121	30 0	36.1	47.0
7	4,160	55,074	90,423	64.0	13.2	21.7
8	13,624	49,493	68,510	33 0	3.6	4.8
9	640	45,984	44,801	− 3.0	71.8	70.0
10	640	47,525	51,707	9.0	74.3	80.8
11	1,120	57,601	57,664	0.1	51.4	51.5
12	2,880	50,246	91,521	82.0	17.4	31.8
13	1,600	43,266	58,721	36.0	27.1	36.7
14	1,280	49,299	52,770	7.0	38.5	41.2
15	1,120	49,178	60,438	23 0	43.9	53.9
16	800	58,158	65,223	12.0	72.8	81.5
17	720	66,084	70,099	6.0	91.9	97.4
18	640	31,404	26,137	−17.0	49.1	40.8
19	640	52,024	58,023	12 0	81.3	90.7
20	800	49,271	61,708	25.0	61.6	77.1
21	960	50,283	47,906	− 5 0	52.4	49.9
22	960	52,523	49,324	− 6.0	54.7	51.4
23	800	45,601	44,320	− 3 0	57.0	55.4
24	1,120	43,465	52,428	21.0	38.8	46.8
25	4,160	54,588	99,696	83.0	13.1	24.0
26	4,640	43,354	74,793	72 0	9.3	16.1
27	20,480	44,290	113,336	156 0	2.1	5.5
28	1,760	55,605	68,183	23.0	31.6	38.7
29	6,400	51,243	81,985	60.0	8.0	12.8
30	1,280	52,757	51,308	− 3 0	41.2	40.1
31	11,200	50,954	78,571	54.0	4 5	7.0
32	8,480	40,211	70,408	75.0	4.7	8.3
33	12,944	37,100	70,841	91 0	2.9	5.5
34	3,200	26,611	67,769	155.0	8.3	21.2
35	4,960	28,086	59,547	112 0	5.7	12.0
Total......	122,008	1,698,575	2,185,283	28.6	13.9	17.9
1890........	......	1,099,850	....	54 4	....	....
Cicero.......	......	....	14,557	.....	....	....
Morgan Park.	2,000	2,329	3,694	58.6	...	1 85
Blue Island. .	1,280	6,114	8,043	31 6	....	6.28

[1] Includes water surface.

PROPORTION OF MUNICIPAL WATER SUPPLY REACHING SEWERS

It is natural to think of sewage as consisting of the water supply defiled by the wastes of the community, in which case the quantity of water consumed would be an accurate measure of the quantity of sewage produced. This impression, however, is incorrect, as only a portion of the municipal water supply reaches the sewers and this may constitute less than half of the sewage because water from other sources also goes into the sewers.

A considerable part of the water supply used by railroads, by manufacturing establishments and power plants, in street and lawn sprinkling, for extinguishing fires, and by consumers not connected with the sewers, fails to reach the sewers and there is usually considerable leakage from mains and service pipes. The Milwaukee Sewage Disposal Commission estimated in 1911 that the quantity of water supply for the several purposes listed in Table 44, never reached the sewers. This is a total of 40 gal., or 38 per cent., of the supply at the time, which was 105 gal. per capita daily.

TABLE 44.—ESTIMATED QUANTITY OF WATER SUPPLIED AND NOT REACHING THE SEWERS, IN MILWAUKEE, 1911
(Gallons per Capita Daily)

Steam railroads	5
Manufacturing and mechanical purposes	5
Street sprinkling	5
Lawn sprinkling	2½
Consumers not connected with sewers	7½
Leakage from mains and services	15[1]
Total	40

[1] The leakage probably greatly exceeds this in many cities.

TABLE 45.—PERCENTAGE WHICH THE FLOW OF SEWAGE WAS OF THE CONSUMPTION OF WATER IN VARIOUS CITIES DURING SUCCESSIVE YEARS

	Mass. No. Met Sewerage District	Worcester, Mass.	Brockton, Mass.	Quincy, Mass.	Providence, R. I.
1900	....	155	59	....	151
1901	......	109	66	....	156
1902	......	158	56	....	153
1903	......	173	60	....	148
1904	127	112	56	130	144
1905	116	124	....	105	150
1906	123	162	73	117	130
1907	126	166	66	120	114
1908	120	163	69	123	120
1909	129	164	....	143	125
1910	....	140	63	143	....
1911	....	145	65	....	....

It is probably true that in many places some of the leakage from mains and services ultimately finds its way into the sewers by infiltration but it is impossible to determine the proportion and it will vary greatly in different communities. In spite of the fact that all of the municipal water supply does not reach the sewers, it is important to know its quantity and to use the data in forming an estimate of the quantity of sewage which will be produced, particularly during the dry season of the year. That the water supply is a very important function of the flow of sewage is indicated by Table 45. It will be seen that although the relation between the two varies widely in different cities, the relation is a fairly constant one in the same city from year to year.

Rate of Consumption in Different Parts of a City.—The consumption of water, and consequently the amount reaching the sewer, varies greatly in different districts of a city. The total amount of water delivered is made up of the requirements for public, domestic and industrial uses and an amount which is usually termed "waste," although "unaccounted for" water might be a better term. Water used for manufacturing was found in 1904 in the Massachusetts Metropolitan Water District to vary in different communities from almost nothing to 24.9 gal. per capita of population. James H. Fuertes estimated the amount used for manufacturing to range from 0.4 gal. per capita in the residential town of Wellesley, to 81 gal. in Harrisburg, as given in Table 46, from his report to the Merchants' Association of New York on the future water supply of that city. It must be remembered that these figures are based on the total population of the city, and that if all the manufacturing is concen-

TABLE 46.—SUBDIVISION OF CONSUMPTION INTO VARIOUS USES
(Gallons per Day per Capita)
(James H. Fuertes, Report on Waste of Water in New York, 1906)

Place	Year	Consumers' use			Public uses	Not acctd. for	Total cons.	Per cent. unacctd. for	Services metered, %
		Mfg.	Domestic	Total					
Brockton...	1904	5 1	15.5	20.6	3.0	13.3	36.9	36	91
Boston.....	1892	30 0	30.0	60 0	3.0	32.0	95.0	34	.....
Cleveland...	1904	40.0	26.0	66.0	10.0	20.0	96.0	21	49
Fall River..	1902	. .	.....	23.4	8.3	8.7	40.5	21	95
Hartford....	1904	3 0	30.0	33.0	5 0	24.0	62 0	39	99
Harrisburg..	1904	81 0	30.0	111.0	5.0	30.0	146 0	21	75±
Lawrence...	1904	8 0	17.0	25.0	5.0	12.0	42.0	29	87
Milwaukee..	1904	45 0	25.0	70.0	5.0	14.0	89.0	16	79
Madison....	1904	......	......	21.0	13.0	37.0	71.0	52	96
Syracuse....	1904	39.3	31.0	70.3	18.0	20.0	108.3	19	72
Taunton....	1904	14 7	21.5	36.2	3.0	24 8	64.0	39	45
Wellesley...	1904	0.4	28.6	29.0	2.5	23.5	55.0	43	100
Yonkers...	1904	24.0	20.0	51.5[1]	2.0	40.5	94.0	43	100

[1] Total includes 7.5 gal. per cap. per day passed through meters at special rate.

TABLE 47.—WATER CONSUMPTION PER CAPITA IN HOUSES OF DIFFERENT CLASSES, 1910 OR 1911

(Journal of the New England Water Works Association, March, 1913)

City	Apartment houses			First-class dwellings			Middle-class dwellings			Lowest-class dwellings		
	No. of houses	No. of persons	Gal. per day per capita	No. of houses	No. of persons	Gal. per day per capita	No. of houses	No. of persons	Gal. per day per capita	No. of houses	No. of persons	Gal. per day per capita
Baltimore, Md.					...	...	20	126	54	25	84	16
Boston, Mass.	50	2,164	37	40	400	60	50	750	33	50	750	15
Boston, Mass.[1]		...	...	...	...	...	...	...	...	50	7,000	24
Cambridge, Mass.	50	1,242	37	50	250	37	50	300	11	50	250	17
Canandaigua, N. Y.	50	218	62	50	290	68	50	180	42	50	146	10
Denison, Tex.	...	...	...	50	153	15	200	799	12	500	3,090	4
Fall River, Mass.	...	...	...	60	328	63	60	457	26	60	1,394	17
Hartford, Conn.	19	560	55	114	659	67	135	1,186	27	98	1,842	24
Hartford, Conn.	75	1,247	24	148	749	43	...	...	...	...	...	...
Holyoke, Mass.	20	2,215	46	15	92	50	20	113	43	...	...	...
Holyoke, Mass.	47	2,118	69 = (apartments with stores)									
Pawtucket, R. I.		...	...	...	...	...	482	4,095	26	766	7,188	12
Pawtucket, R. I.	...	...	...	...	...	...	...	...	...	441	4,534	12
Peoria, Ill.	5	150	84	5	30	74	20	80	32	5	15	11
Peoria, Ill.	5	200	63	13	104	47	25	125	28	8	40	6
Plymouth, Mass.	...	...	...	23	94	47	15	67	33	2	4	14
Washington, D. C.	101	3,470	135	81	500	75	100	400	30	100	500	37
Wilmington, Del.	25	500	73	25	189	73	25	125	44	...	...	...
Worcester, Mass.	50	1,875	60	50	277	42	50	385	66	50	1,179	12
Totals	497	15,989	...	727	4,115	...	1,302	9,188	...	2,258	28,016	...
Averages	...	...	62	...		54	...	...	31			15

[1] Lowest-class dwellings, lower figures = those for tenement blocks containing from 15 to 30 families each.

trated in one portion the per capita consumption figured on the basis of the population of that district would be very much higher. The quantity used for manufacturing depends entirely on the character and amount of the industries, and whenever possible an actual canvass and estimate of quantities should be made.

The amount used for domestic purposes varies with the class of residence, first-class residences with many fixtures using more per capita than the less elaborate houses, as shown in Table 47.

In some of the largest cities where considerable districts are almost entirely devoted to business and the number of people in the district during the day, but resident elsewhere, is very large, per capita figures of consumption must be studied with great care before any conclusions are drawn from them. The figures in Table 48 illustrate this clearly. The subject was investigated by the Metropolitan Sewerage Commission of New York which reported in 1910 that the actual resident population of the Borough of Manhattan was increased about one-third daily by

TABLE 48.—CONSUMPTION OF WATER IN SECTIONS OF MANHATTAN
(W. W. Brush, Proceedings Am. Water Works Assoc., 1912)

Characteristics of district	Consumption, million gal. per day	Resident population	Consumption per capita, gal. per day
Gagings of 1902–03			
Large hotels, high-class residences	1 87	8,396	223
East Side tenements....	1 44	38,906	37
East Side tenements ...	5 40	90,000	60
Residence and high-class apartments.	0.76	10,164	75
Business, office buildings, waterfront, shipping.	9.45	11,000	860
High-class apartments and hotels	1 37	8,872	154
Uptown residences and medium-class apartments.	4 89	4,380	112
Upper East Side tenements, waterfront, power houses and breweries	2.75	39,969	69
Gagings, 1911			
East Side tenements, some waterfront.	11.44	230,500	50
All classes..................	29.48	204,557	144
High-class apartments and residences.	22.18	186,990	118
High-class apartments, residences and tenements.	12.74	138,800	92
East Side tenements and waterfront.	8.28	84,580	98
High-class apartments, residences, tenements and waterfront.	14.82	173,000	86
All classes.....	13.38	169,100	79
All classes...	13 66	209,393	65

TABLE 49.—RESIDENT AND TOTAL POPULATIONS OF CERTAIN DISTRICTS IN MANHATTAN, 1903 (HILL)

District	Resident population	Total population	Increase of total over resident population	Character of district
1	8,396	12,156	45 per cent.	Residential and high-class hotel
2	38,906	38,906	0 per cent.	Tenement houses.
3	90,000	90,000	0 per cent.	East Side tenements.
5	32,200	32,450	1 per cent.	Moderate priced apartments.
6	10,164	10,164	0 per cent.	Apartment houses; private houses.
7	3,076	6,076	98 per cent.	Gas works; large shops; railroad yards.
8	11,000	114,000	937 per cent.	Office buildings.
9	8,872	8,872	0 per cent	Apartment houses; private houses.

the influx of persons engaged in business pursuits there but residing elsewhere. The various transportation companies bringing passengers into the borough furnished information to the Commission indicating that 413,500 residents on Long Island, 203,800 in New Jersey, 17,200 on Staten Island and 42,900 north of the Bronx came to Manhattan daily for business purposes. A somewhat earlier investigation was made by Nicholas S. Hill, Jr., while Chief Eng. of the Department of Water Supply, Gas and Electricity of Manhattan; the results are summarized in Table 49, from *Eng. News*, April 9, 1903.

This influx of non-residents, which is the cause of greatly increased flow in the sewers serving such districts, doubtless has a corresponding, though smaller, effect upon the flow in sewers serving the districts in which these persons reside. However, as their residences are widely scattered, it is probable that in no place will the reduction in flow be sufficient to warrant any allowance for it in design, although it is very important to provide for the increased flow in the sewers serving the business districts into which they go.

Water Consumption in Cities.—The consumption of water in American cities, particularly the different classes of consumption and the variations in the hourly, daily, weekly and monthly rates at which water is used, is discussed in detail in a report by Metcalf, Gifford and Sullivan in the *Journal* of the New England Water Works Association, March, 1913, upon which much of the following discussion of the subject has been based. From that source are taken the curves of the percentages of services metered and the per capita water consumption in Worcester, Fall River, and Lawrence, Mass., and Providence, R. I., shown in Fig. 56. In spite of the large proportion of metered services, the quantity of water consumed is seen to have steadily increased in Fall River until in 1910 it was about 50 gal. per capita, a relatively small consumption, however. A similar increase was apparent in Worcester between 1897 and 1904, since which time it has fallen to nearly 60 gal. per capita. In Lawrence, with a steady increase in the proportion of services metered, there was a nearly uniform reduction in the quantity of water used from 1892, when the consumption was slightly in excess of 90 gal., to 1904, when it fell nearly to 40 gal. per capita daily. From 1904 to 1909, however, there was a slight upward tendency. In Providence, R. I., where the water system has been generally supplied with meters for many years, there has been a gradual tendency toward increasing the per capita consumption although in recent years it has exceeded 70 gal. but once.

The immediate effect of largely increasing the proportion of meters is shown by the Minneapolis and Cleveland curves, Fig. 57, in which the drop in consumption following the increase in meters has been very substantial.

In the cities in which but few of the services were metered, and no

serious efforts made to restrain waste, there has been a rather steady increase in per capita consumption. The quantity of water used in the cities which are not well supplied with meters is found, as a rule, to be largely in excess of that in the cities where metered services are general. This is shown in Fig. 58, by the two curves indicating the average

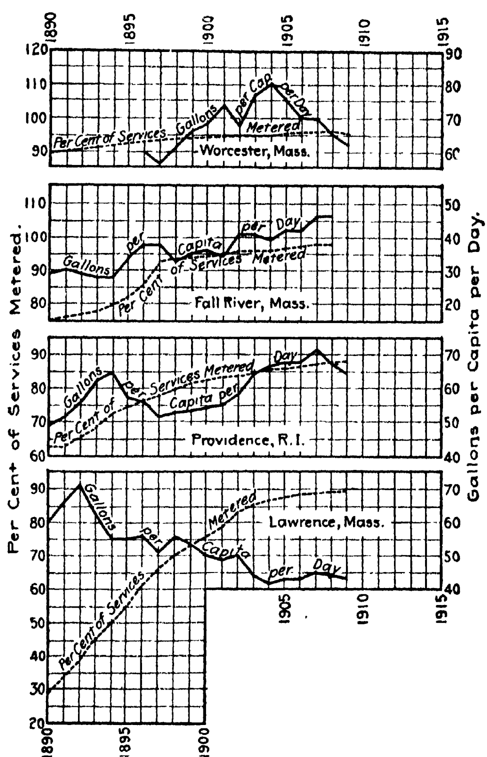


FIG. 56.—Water consumption in Worcester, Fall River, Providence and Lawrence.

per capita consumption in cities having less than 15 per cent. and the corresponding consumption in the cities having more than 50 per cent. of the services metered.

There is some difference of opinion relative to the effect of meters in reducing the quantity of water, some holding the view that if meters are installed the water consumption will be permanently reduced, while

others, acknowledging that the immediate result of installing meters may be a reduction in quantity, believe that the tendency of the times is toward the use of gradually increasing quantities and that the effect of meters in reducing consumption will gradually be offset until the reduction effected is wiped out and the consumption gradually increases beyond that of the time the meters were installed. All admit, however, the tendency of meters to check the waste of water.

There is some foundation for both views. It seems evident that a

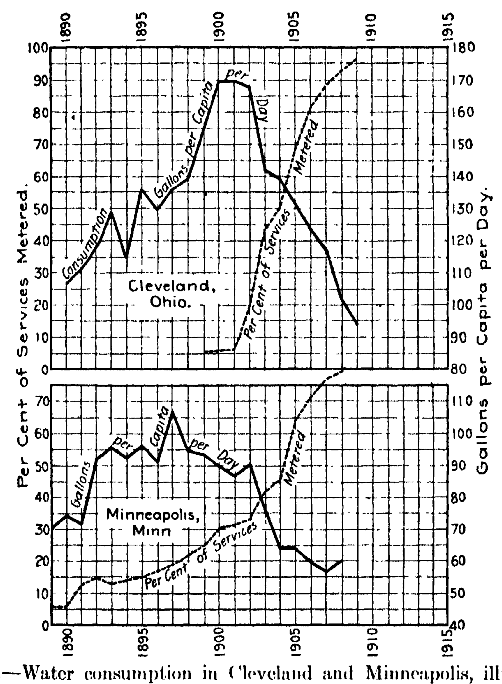


FIG. 57.—Water consumption in Cleveland and Minneapolis, illustrating effect of meters.

thorough system of metering will be an effective instrument in the hands of conscientious and capable management; on the other hand, the mere fact that meters are provided is no guarantee of a low water consumption, and if the meters themselves are neglected and allowed to remain out of repair, or if the facts to be learned from the records compiled from them are not wisely utilized by the officials in charge, they may fall far short of performing their full possible function in holding down the consumption.

It is, of course, desirable that the municipality should provide all the water which its citizens can use to advantage, but there is nothing to be said in favor of the waste of water through neglect of fixtures, faulty pipe lines and services, or surreptitious connections through which large quantities are too frequently drawn without payment therefor or even the knowledge of the proper officials. Such consumption not only increases the burden upon those who pay for providing the water works, but it may also increase the cost of sewerage, particularly where pumping and purification are necessary.

Low water rates tend toward the use of increased quantities of water and may render the meter less effective as an agent in restricting waste, in that the size of the bills due to waste may not be sufficiently large to cause an effort on the part of the consumer to economize in the quantity of water drawn through his fixtures. These observations are

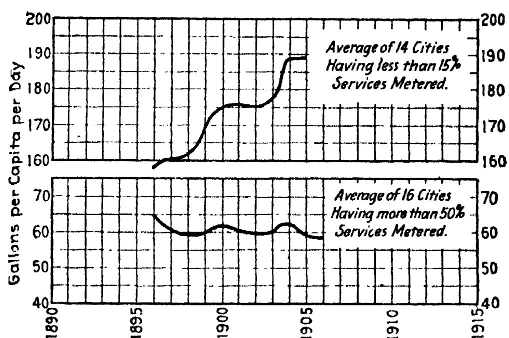


FIG. 58.—Composite curves of water consumption in cities with small and large percentages of services metered.

equally pertinent to rules permitting the use of a large quantity of water at a low minimum rate.

From the statistics available two conclusions seem to be warranted: First, there is a gradual tendency toward increase in the quantity of water used per capita of population. This is undoubtedly due, so far as it relates to domestic uses, to more elaborate plumbing. The number of fixtures per person, as well as the quantity of water required per fixture, has greatly increased in recent years. In the larger cities, the increased consumption may be due in part to the difficulties surrounding the management of the water departments, which are usually much greater than in the smaller cities and towns. Second, the evidence furnished by such cities as Providence, Worcester, Fall River and Lawrence, indicates that with careful management aided, perhaps, by thorough metering, it

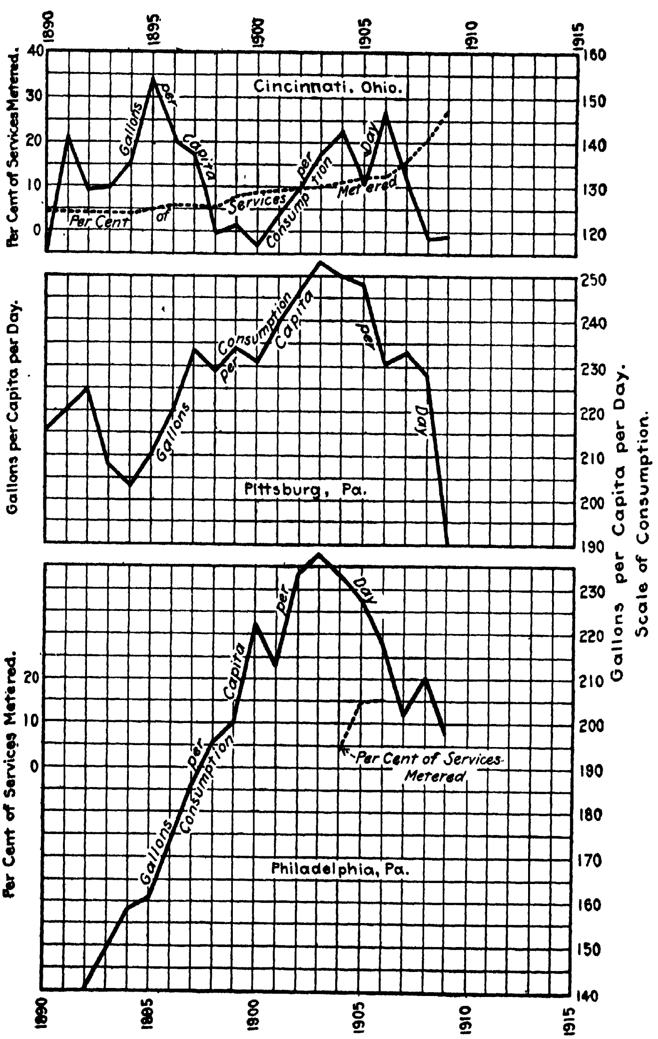


FIG. 59.—Reduction of consumption in Cincinnati, Pittsburg, and Philadelphia.

is possible to hold down the increase in quantity of water consumed to reasonable proportions. This is also well illustrated by the recent reduction in consumption in Pittsburg, Cincinnati and Philadelphia, shown in Fig. 59.

The Milwaukee Sewage Disposal Commission, which studied this question carefully, said in 1910 that, taking into account the history of the Milwaukee water works, the industrial character of the city, the low water rate of 6 cents per 1000 gal., as well as the availability of river and lake water, it was of the opinion that an increase of 5 gal. per capita per decade was a reasonable allowance to make for the next 40 years.

Fluctuations in Water Consumption.—While it is important to know the average quantity of water consumption, it is of still greater value to

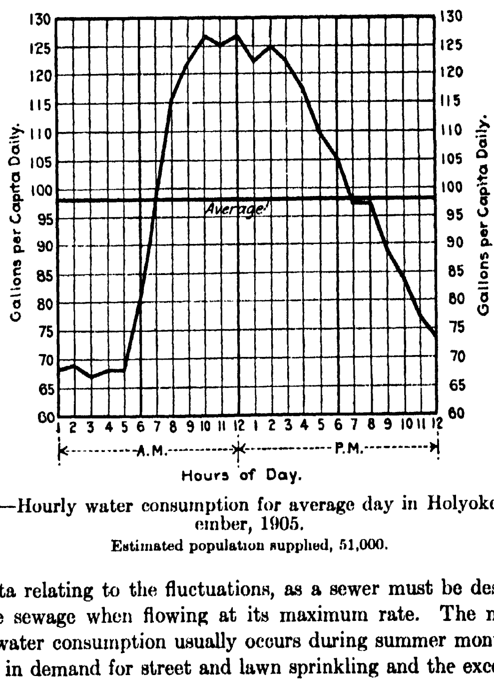


FIG. 60.—Hourly water consumption for average day in Holyoke in November, 1905.
Estimated population supplied, 51,000.

have data relating to the fluctuations, as a sewer must be designed to take the sewage when flowing at its maximum rate. The maximum rate of water consumption usually occurs during summer months when water is in demand for street and lawn sprinkling and the excess is not likely to reach the sewers, or in the winter when large quantities are allowed to run to prevent freezing of pipes and fixtures, this excess usually finding its way into the sewers. In Table 50 have been compiled records of maximum water consumption for 67 Massachusetts cities and towns (1910) taken from report of Committee on Water Consumption,

TABLE 50.—RECORDS OF MAXIMUM WATER CONSUMPTION FOR MASSACHUSETTS CITIES AND TOWNS, 1910

(By courtesy of X. H. Goodnough)

City or town	Population	Average daily consumption per person	Max. monthly consumption: Gal. per person per day	Max. monthly consumption: Per cent. of average for year	Max. weekly consumption: Gal. per person per day	Max. weekly consumption: Per cent. of average for year	Max. daily consumption: Gal. per person per day	Max. daily consumption: Per cent. of average for year
Abington and Rockland	12,383	45	63	137	70	151	90	197
Amesbury	9,894	44	50	114	51	116	68	155
Andover	7,301	86	99	115	....	......	162	189
Attleborough	16,215	54	62	115	63	116	91	169
Avon	2,013	36	55	153	72	200	102	283
Ayer	2,797	50	64	128	72	144	172	342
Beverly	18,650	91	146	160	191	210	224	246
Braintree	8,066	81	87	107	93	115	108	133
Bridgewater and E. Bridgewater	11,051	22	27	123	29	132	39	177
Brockton	56,878	39	45	115	55	141	69	177
Brookline	27,792	89	103	116	117	132	128	177
Cambridge	104,839	100	106	106	111	111	119	119
Canton	4,797	61	75	123	84	138	97	159
Danvers and Middleton	10,536	89	108	121	136	153	158	178
Dedham	9,284	129	153	119	163	128	182	141
Easton	5,139	24	28	117	38	158	63	263
Fall River	119,295	44	47	107	50	114	54	123
Foxborough	3,863	50	51	102	59	118	75	150
Framingham	12,948	48	58	121	65	137	86	179
Franklin	5,641	61	88	144	96	158	127	208
Gardner	14,699	44	49	111	55	125	108	246
Gloucester	24,398	55	96	156	114	207	130	237
Grafton	5,705	18	22	122	24	133	32	178
Hudson	6,743	49	60	123	65	133	......	.....
Ipswich	5,777	42	60	143	84	200	106	253
Lawrence	85,892	45	51	113	60	133	60	133
Lowell	106,294	51	57	112	66	129	75	147
Lynn and Saugus	97,383	72	79	110	87	121	108	150
Manchester	2,673	120	261	217	327	271	363	302
Mansfield	5,183	75	97	129	103	137	276	368
Marblehead	7,338	79	147	186	169	214	187	237
Marlborough	14,579	37	42	114	59	159	80	190
Maynard	6,390	36	39	108	47	130	60	167
Methuen	11,448	38	54	142	67	176	69	182
Middleborough	8,214	42	53	126	65	155	90	214
Milford and Hopedale	15,243	51	60	118	64	125	71	139
Montague and Erving	8,014	66	75	114	70	106	153	232
Nantucket	2,962	67	128	191	154	230	176	263
Natick	9,866	57	70	123	82	144	170	298
Needham	5,026	66	88	133	98	148	119	180
New Bedford	96,652	81	88	109	98	121	106	131
Newburyport	14,949	68	83	122	94	138	121	178
Newton	39,806	63	74	118	82	130	95	150
North Andover	5,529	40	53	132	64	160	78	195
North Attleborough	9,562	52	73	140	82	158	95	183
North Brookfield	3,075	66	81	123	112	170	213	319
Norwood	8,014	63	86	136	86	136	132	211
Orange	5,282	26	34	131	41	158	62	182
Peabody	15,721	168	198	118	182	108	270	161
Plymouth	12,141	103	131	127	140	136	171	166
Provincetown	4,369	38	69	182	77	203	93	245
Randolph and Holbrook	7,117	74	120	162	140	189	175	237
Reading	5,818	35	52	149	60	172	66	189
Rockport	4,211	72	148	205	196	272	212	295
Salem	43,697	90	101	112	103	114	133	148
Sharon	2,310	57	97	170	120	210	137	240
Stoughton	6,316	35	43	123	58	166	75	214
Taunton	34,259	63	70	111	74	118	87	138
Wakefield	11,404	61	85	139	107	175	127	208
Walpole	4,892	102	119	117	149	146	252	247
Waltham	27,834	88	95	108	98	111	108	123
Webster	11,509	38	49	129	53	139	72	189
Wellesley	5,413	61	68	111	79	129	112	184
Whitman	7,292	29	42	145	44	152	......	......
Winchendon	5,678	30	35	117	40	133	45	150
Woburn	15,308	139	172	124	190	137	232	167
Worcester	145,986	74	85	115	......	......	103	139
Average	....	63	81	128	93	147	123	198

Statistics and Records, published in New England Water Works *Journal* for March, 1913. The average water consumption in the cities and towns included in this compilation was 63 gal. per capita per day. The average maximum monthly consumption, the maximum weekly consumption and the maximum daily consumption were 128, 147 and 198 per cent. of the average daily consumption for the year, respectively. There were, however, instances in which the maximum rates greatly exceeded these averages. For example, in Manchester and Mansfield, Mass., the maximum daily consumption was 302 and 368 per cent. of the average for the year, respectively. These high rates of flow, however, almost always occur at times when the usual proportion of the flow does not reach the sewers, as in the most dry portion of the summer, or in winter

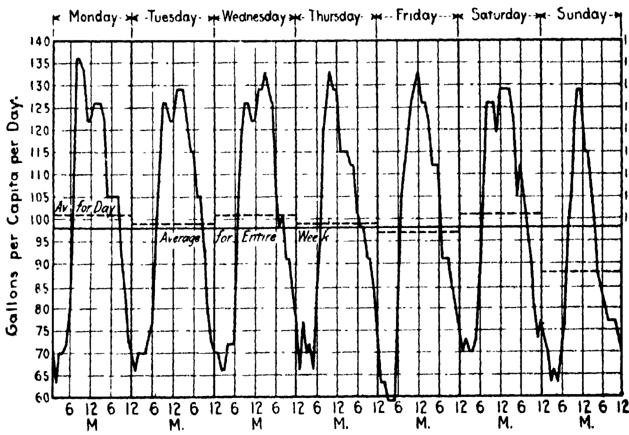


FIG. 61.—Fluctuations in water consumption in Holyoke during week ending November 17, 1905.

when water from other sources, as for example, ground water, is likely to be at a minimum.

In addition to the fluctuations in flow already discussed, there is an important variation from hour to hour each day, as illustrated by Figs. 60 and 61 taken from the same report. (*N. E. W. W. Jour.*, March, 1913.) It will be seen from Fig. 61 that the maximum peak flow during the week occurred on Monday, when the draft was about 135 per cent. of the average for the day, and the minimum peak draft was on Sunday when it was 146 per cent. of the average for the day, these rates being 139 per cent, and 132 per cent. respectively, of the average rate of draft for the week.

TABLE 51.—RELATION OF ANNUAL AVERAGE QUANTITY OF SEWAGE TO WATER CONSUMPTION. NORTH METROPOLITAN SEWER DISTRICT, BOSTON, MASS.

Year	Precipitation in inches at Chestnut Hill	Based on total population of district			Based on population *connected* with sewer system	
		Average sewage flow, gallons per capita per day	Average water consumption, gallons per capita per day	Ratio of sewage flow to water consumption, per cent.	Average sewage flow, gallons per capita per day	Ratio of sewage flow to water consumption, per cent.
1904	43.40	121.6	100 3	121.2	155 7	155.3
1905 ...	40.84	113.5	101.9	111.3	139 3	136.6
1906 ...	47.16	118.4	99.8	118.7	149 8	150 2
1907....	51.83	128.2	106.1	120.8	151.7	142 9
1908 ...	43.31	116.8	104.9	111.3	137.8	131.3
1909......	47.62	115.9	94 7	122.4	133 9	141 4
1910...	39.05	110.3	92.3	119.4	126.8	137 4
1911......	41.28	96.9	86.9	111.5	110.0	126.6
1912........	39.96	100.2	86.8	115.4	112 5	129.7
Average..	43.83	113.5	97.1	116.9	135.3	139.0

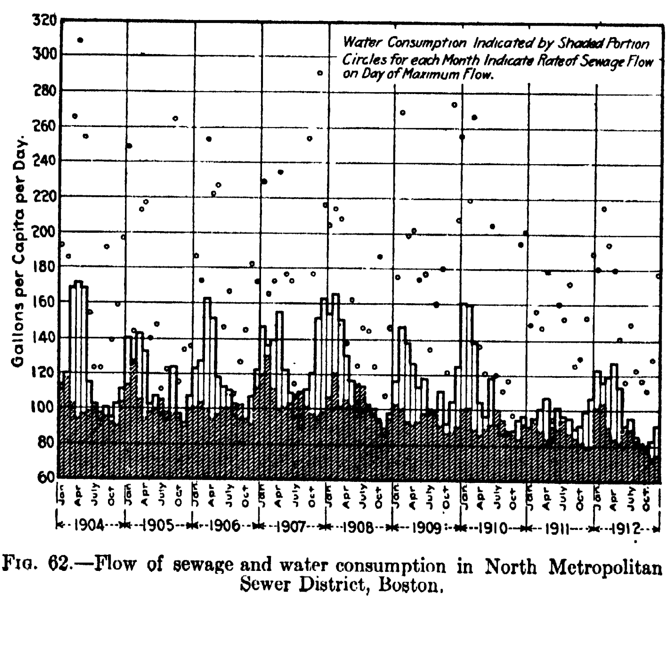


FIG. 62.—Flow of sewage and water consumption in North Metropolitan Sewer District, Boston.

The hourly fluctuation in rate of water consumption has a decided effect upon the rate of sewage flow, as discussed later in this chapter. It is not, however, entirely responsible for the fluctuation in the rate of flow of sewage, for in some places large quantities of ground water are pumped by industrial establishments and discharged into the sewers during the working hours of the day, thus tending to increase the peak flow beyond the amount resulting from the normal fluctuation in the draft on the municipal water supply.

Allowing for water drawn from private supplies, the peak rate of consumption during the day of maximum use may be taken at 150 per cent. of the average draft upon the municipal supply for that day. This rate, however, will vary in different places.

If this peak consumption is applied to the maximum draft for a single day of 198 per cent. of the average annual consumption, and it is assumed that the portion of the annual consumption which finds its way into the sewers averages 50 gal. per day, we have a maximum rate of contribution from the public and private water supplies of about 150 gal. per capita daily ($50 \times 1.98 \times 1.50 = 148.5$). This will serve to illustrate the theory of the yield of sewage based on water consumption, but should not be applied in design unless local conditions are found to warrant it.

Ratio of Sewage Flow to Water Consumption.—The North Metropolitan sewerage system of Boston furnishes valuable information regarding the relations between the quantity of sewage reaching a large intercepter, and the population, area, water consumption and rainfall of the district served. The relations between the quantity of sewage and the water consumption are given in Table 51, and the relations of the maximum to the average monthly sewage flow and to the average monthly water consumption are given in Fig. 62. The circles representing rates of flow on days of maximum flow must not be misinterpreted, for the sewer is protected by storm outlets, which permit the discharge of much of the flow, unmeasured, at such times.[1] The relations between the sewage flow and the water consumption for the dry period of each year from 1904 to 1912 are given in Table 52. The dry months were selected in the driest season of the year and after a month of dry weather; in a few cases they show a rather high rainfall, but it was concentrated in a few

[1] These average annual figures are open to the criticism that they include some storm water, for some of the local sewers discharging into the intercepters are on the combined system. Nevertheless, the figures are fairly representative of the sewage flow as influenced by infiltration. As shown later, it is estimated that during dry weather, if no storm water enters the sewers, the sewage flow will be about 91 per cent. of the water consumption. The circles in Fig. 62 representing the discharge on days of maximum flow do not give the maximum volume of sewage and storm water, for the overflows may have been in operation.

days so as to leave the month a dry one as a whole. It will be noticed that there are no figures for 1911; none of the monthly records in that year is representative of dry weather conditions. In fact the 1908 and 1909 figures are probably high on account of some rain-water in the sewage, the rainfall figures in the table making this appear very probable.

TABLE 52.—RATIO OF SEWAGE FLOW TO WATER CONSUMPTION DURING DRY WEATHER. NORTH METROPOLITAN SEWER DISTRICT, BOSTON, MASS.

Year	Month	Rainfall (inches)		Average sewage flow, gallons per capita per day	Average water consumption, gallons per capita per day	Relation of sewage flow to water consumption, per cent
		For month	For month previous			
1904	August.....	2.74	1.48	93.8	97.8	95.9
1905	August.....	3.47	1.92	94.0	98.3	95 6
1906	September..	2.92	1.82	95.2	101.6	93 8
1907	August.. .	1 79	1 49	88.7	110.2	80.6
1908	October....	4.34	1.22	93.3	95 9	97.2
1909	August....	4.11	1.10	98 8	97.5	101.2
1910	August..	1 18	1.93	86 9	94.4	92.1
1910	September.	2.65	1.18	86.3	88.7	97.2
1912	September..	1 72	2.24	80 0	83 4	95.9
1912	October.	1.61	1 72	76 9	81 1	94.9
Average..	......	2.65	1.61	89.4	94.9	94.4

The average relation of dry weather sewage to water consumption is given in Table 52 as 94.4 per cent. This would probably be reduced by about 3 per cent. if it were possible to exclude storm water entirely from the sewage flow. The correction of 3 per cent. was obtained from the graphical study by the authors of detailed record sheets of the Metropolitan Sewerage Board for seven typical months during 1904 to 1909 inclusive. From a study of the diagrams of sewage flow during each month, it seems probable that the true ratio of sewage flow to water consumption is about 90 per cent., if the entire amount of storm water is excluded. If this estimate is in error, it is probably too high, as the quantity of ground water assumed in rounding off the figures is only about 1200 gal. per mile of sewer, this being in extremely dry weather. This ratio, it must be kept in mind, is a purely local one and can hardly be expected to agree with other conditions than those on which it is based. Unfortunately very little information of this nature is available, and engineers having opportunities to keep such records should not neglect to do so.

The ratio of 90 per cent. does not mean literally that 90 per cent. of

the water supply is delivered to the sewer, but rather that the dry-weather flow of sewage bears that relation to the supply.

ADDITIONS TO THE SEWAGE

A certain amount of sewage reaches the sewers from those hotels, public baths and other buildings which supplement the public water supply with water from wells. In addition to this uncertain influence on the sewage flow of a metropolitan district, there are two much more important sources of additions to the sewage, viz., ground water and industrial wastes originating from the use of water derived from private sources.

Ground Water.—For sewerage purposes, ground water is a term which includes not only all water in the pores of the materials through which sewers are laid, but also the surface water leaking into sewers through perforated manhole covers and defective manhole masonry. Where the sewers are on the combined plan, ground water also includes the dry-weather flow of any small brooks connected with the system. From a half to three-fourths of the rainfall usually runs off very quickly into the storm-water drains or combined sewers, when there are any, and the remainder percolates into the ground, becoming ground water. The filtration of water from rivers, lakes, and tidal waters through the ground sometimes has considerable effect on the height of the ground-water table. In such localities, allowances should be made for leakage into sewers, and it is desirable to construct the sewers when the lakes or rivers are low and at low water between tides to avoid expense and trouble due to very wet trenches.

The elevation of the ground-water table rises and falls continually, and its fluctuations were formerly held by a large number of sanitarians to be the cause of typhoid fever. This was known as the von Pettenkofer theory and is no longer held to be true, except by a very few persons. However, the presence of large amounts of ground water in the earth about the sewers results in leakage into them, which causes a serious problem where the expense of disposing of the sewage is heavy. It is a wise policy for the engineer to neglect no opportunity to acquire information regarding the phenomena presented by the flow of underground water.

The sewers first built in a district usually follow in a general way the natural water courses, and therefore lie in the bottoms of the valleys. Such sewers are often, especially in case of combined systems, built very close to, or actually in, the natural beds of brooks. They are not usually extended to the extreme upper end of the district at first, and consequently the natural run-off through these brooks is taken into the sewers. Such brooks frequently flow with gradually diminishing volume

for many days after the immediate run-off from a storm has passed by, and perhaps even throughout the dry season. The flow during the remainder of the time until the next storm is made up of the water draining out of the land, and is therefore logically classed with ground water and, as its flow is continuous though gradually diminishing, it has the same effect upon the quantity of sewage. As a result of these conditions, such sewers receive comparatively large quantities of ground water, while it is but natural to expect that sewers built in these districts in later years, necessarily at higher elevations, will receive smaller quantities of leakage and brook flow. Moreover, as the paved and built-over area increases the water falling upon the surface runs off more rapidly through the water courses, drains, or combined sewers, and leaves less to percolate gradually through the ground and thus to find its way into the sewers by infiltration or leakage.

Many measurements have been made to determine the quantity of ground water which finds its way into sewers. The results of these observations indicate that the maximum quantity of infiltration may be as low, under the most favorable conditions, as 5000 or 10,000 gal. per day per mile of sewer. On the other hand, they show that the leakage sometimes amounts to from 20,000 to 40,000 gal. per day per mile of sewer, and at times of very high ground water, or during rain when there is leakage through manhole covers, even in separate systems, it may run as high as 100,000 gal. per day per mile of sewer. In fact there are instances where leakage has materially exceeded this quantity.

As a rule, there has been a growing tendency toward securing as nearly water tight construction as possible, and it may be true that the older systems receive greater quantities of ground water than some of the better constructed modern systems.

Leakage.—The amount of ground water which finds its way into the sewers is called "leakage." It is a very variable part of the flow in the sewers, depending on the quality of the materials and workmanship employed in the original construction, on the degree of care in maintenance and in preventing damage to the sewers by drain layers or plumbers when connecting house drains, and on the height of the ground-water table.

In the case of the North Metropolitan (Boston) intercepter, already mentioned several times in this chapter, it is possible to form a fairly close estimate of the amount of this leakage, for if 90 per cent. of the average monthly water consumption is equivalent to the sewage flow at the same time, by subtracting this quantity from the measured sewage flow, the remainder will be the infiltration into the sewers. As this leakage will be greatest in very wet weather, the figures for the most wet period of each year have to be studied, and the results of such a study in this case are given in Table 53.

TABLE 53.—LEAKAGE IN NORTH METROPOLITAN SEWER DISTRICT, BOSTON, IN APRIL AND MAY

	Average	Maximum	Minimum
Gallons per capita per day.	62.2	93.8	38.7
Gallons per acre per day.	1,738	2,577	1,094
Gallons per mile of sewer per day. . .	50,600	78,900	30,900

The amount of leakage is stated in different ways by different engineers as so much per unit length of pipe, per capita or per acre. It depends, of course, on the length of pipe, and to a certain extent on the population, which affects the number of connections and the lengths of the sewers and the consequent opportunity for leaks. In Table 54 are shown the allowances made for leakage in the designs for various cities and in Table 55 the actual measurements of leakage at certain places.

A paper on the "Infiltration of Ground Water into Sewers" by John W. Brooks (Trans. Am. Soc. C. E., vol. lxxvi, 1913) enumerates the factors influencing infiltration, as follows: 1. The diameter and length of the sewer; 2. the material of which the sewer is constructed, and (*a*) in vitrified pipe sewers, the type of joint used, (*b*) in concrete or brick sewers the type and quantity of waterproofing used; 3. the skill and care used in laying the sewer; 4. the character of the materials traversed by the sewer; 5. the relative positions of the sewer and the ground-water level. After discussing the various units, such as gallons per day per capita or per mile of pipe, he suggests the following units: For vitrified pipe, gallons per day per foot of joints; for concrete and brick sewers, gallons per day per square yard of interior surface.

In the discussion of the paper, John H. Gregory suggested as a unit the number of gallons per day per inch of diameter per mile of sewer. S. L. Christian stated that observations practically checked previous assumptions as to the quantity of ground water to be provided for at New Orleans where all of the sewers are below the ground water level. He stated that the leakage in gallons per day per mile of sewers was as follows: 1907, 55,000; 1908, 53,000; 1909, 51,000; 1910, 51,000, 1911, 48,000; 1912, 42,000. E. G. Bradbury questioned the value of a unit based on the diameter of the sewer, as but very few sewers are sufficiently watertight to prevent the lowering of the ground-water in the vicinity to the level of the pipe. He was of the opinion that most sewers permit the entrance of ground water about as fast as it gets to them.

In general the authors have found that water finds its way into sewers through defective joints in pipes or brick structures, through concrete which is porous and through cracks due to contraction or other causes. These imperfections are sufficiently numerous and large to allow the infiltration of water to such an extent that the water table at the sewer rarely lies above its crown and usually is found near the invert, although

TABLE 54.—BASIC QUANTITIES PER DAY USED IN FIXING THE SIZE OF INTERCEPTING SEWERS IN VARIOUS CITIES

City	Ultimate		Av. density of population	Est. water consumption, gal. per cap	Av. flow gal per cap.			Storm water			Max rate dry weather flow of sewage gal. per cap.	Total provision		Date of	
	Area in acres	Population			Domestic sewage	Mfg wastes	Ground water	In	Gallons Per cap.	Gallons Per acre		Gal. per capita	Gal. per acre	Design	Ultimate estimate
Chicago.															
No. Lke. Front majority rept	3,020.0	120,800	40 0	...	...	...	...	6.0	4,073	162,910	180	4,282.0	171,270	1897	...
Minority rept.	3,020 0	120,800	40 0	...	...	...	...	2 0	1.360	54,400	180	1,540.0	61,600	1897	...
So. Lke. Front majority rept.	7,401 0	...	[3]	...	...	...	...	[4]	...	...	180	...	166,890	1897	...
Minority rept.	7,401.0	...	[3]	...	...	...		[5]	...	...	...	...	...	1897	...
Boston, Mass[1]	9,600 0	600,000	62½	75	75		...	0.24	108	6,730	150	258 0	16,125	1877	...
Massachusetts No. Metrop intercepter.	46,000.0	513,000	11.1	...	...	...	...	...	...	...	...	239[6]	2,670	1889	1930
Metrop. Chas riv. intercept	26,900.0	157,000	5.9	...	...	...	...	...	...	...	...	224 0	1,316	1889	1930
Metrop. Neponset riv. valley intercepter.	21,864.0	213,316	9 8	...	...	25 max	...	...	...	...	...	223.5	2,190	1895	1930
Metrop. high level sewer.	64,600.0	986,000	15 2	140	...	...	24 max.	...	...	...	200	300 0	4,580	1899	1940
Baltimore	...	1,000,000	...	...	...	...	...	...	...	...	...	300.0[7]	...	1906	1925 to 35
Providence	11,351.0	337,000	29.7	...	60	42 max	...	0 24	218	6,470	145	363.0	10,780	...	...
Paterson[2]	...	300,000	...	75	75	18 max.	14	...	...	...	...	250.0	...	1906	1938
Louisville	7,829 0	263,550	33 7	...	100	57	58	...	...	...	...	357 0	12,000	1906	...
Milwaukee (1889)	15,433 0	562,010	36 4	100	...	...	...	0.24	178	6,480	...	419 0	15,240	1889	1935
Passaic Valley sewer project	61,827.0	1,649,440	27.1	...	100	38 max	29 max	...	...	...	...	241.0[8]	6,422[8]	1908	1940
Milwaukee (1910)	35,000.0	850,000	22.8	105 to 125	70 to 85[9]	57 2	75 to 81	...	...	...	248 to 271	350.0	8,500	1910	1950
New Bedford	8,808.0	220,695	25.1	...		...	...	...	...	...	125	400 0	10,000	1911	1940
Fitchburg, Mass	5,858.3	82,160	12 3	.	150	81	139	...	...	...	...	432 0	5,180	1911	1940
Syracuse	10,933.3	410,000	37 5	90	...	...	...	...	...	...	125	375.0	14.063	1895	revised 1907
Fitchburg, Mass	8,134.4	87,200	10 7	100	75	183 max.	115 max.	...	...	...	...	450.0	4,800	1912	1940
Fort Wayne, Ind	12,160 0	150,000	12 3	100	150	...	160	...	...	...	150	350 0	4,310	1911	1950

NOTE: The foot-notes will be found at the bottom of page 185.

Table 54 was prepared from the following sources:

Chicago.—From Report of Chicago Comn., 1897.

Boston.—Main Drainage Works of Boston and its Metrop. Sewerage District (p. 8), published under authority of Metropolitan Sewerage Comm'rs. (1899).

Mass., North Metrop. Interceptor.—Report of Mass. State Bd. of Health, Drainage of Mystic and Charles River Valleys (1890).

Mass., Charles River Valley Interceptor.—Idem.

Mass., Neponset River Valley Interceptor.—Report Metrop. Sewerage Comm'rs. (1895) p. 38.

Mass. Metrop. High Level Sewer.—Report Metrop. Sewerage Comm'rs. High Level Gravity Sewer (1/1899), p. 44.

Baltimore.—Baltimore Sewerage Comn. (1906), p. 23.

Providence.—Furnished by City Engineer.

Paterson.—Report, Joint Comn. on Sewage Disposal (1908), p. 111.

Louisville.—Furnished by H. P. Eddy.

Milwaukee, 1889.—Report of Comn. of Engineers.

Passaic Valley Sewer Project.—Report Passaic Valley Sewerage Comm'rs. (1908), p. 7.

Milwaukee (1910).—Report of Comn. of Engineers.

New Bedford.—Report of Metcalf and Eddy, 1911.

Fitchburg.—Report of Metcalf and Eddy, 1911.

Syracuse.—From *Eng. News* (July 13, 1911, p. 38).

Fort Wayne, Ind.—Report of Metcalf and Eddy, 1911.

[1] Main drainage figures are with sewers 1/2 full. The slope and size of the outlet section of intercepter indicates a capacity, flowing full, of 407 gal. per capita. [2] See Paterson report for explanation of figures. Area taken from census report and may vary somewhat from actual areas used in design. [3] Density, 60 in district tributary to 12th and 22nd St. sewers; 70 in district tributary to 35th St. sewers; 60 in district tributary to 41st and 45th St. sewers. [4] Storm water, 0.273 in. per hr., area north of 39th St; 0.25 in. per hr., area trib. to 41st and 45th St. sewers; 0.22 in. per hr., area trib. to 51st St. sewer; 0.16 in. per hr., area trib. to 56th St. sewer. [5] Storm water, 0.273 in. per hr. 12th and 22nd St. sewer; 0.15 in. per hr. 35th St. sewer; 0.0833 in. per hr. 39th St. sewer; 0.0625 in. per hr. 63rd St. sewer. [6] 224 gal. in districts provided with separate system of sewers; 261.8 gal. in districts provided with combined system of sewers. [7] Pumping plants and parts easily duplicated 150 gal. per day. Disposal plant 75 gal. per cap. per day. [8] Passaic Valley figures are for sewer flowing full. Figures in report assume maximum flow with sewer 3/4 full. [9] Water supply reaching sewers.

its elevation varies greatly with the quantity of rain and snow water percolating into the ground. This is usually greatest in the northern part of the country in the spring of the year, when the frost coming out of the ground leaves it porous so that the water from slowly melting

TABLE 55.—LEAKAGE OF GROUND WATER INTO SEWERS

Place	Gal. per day per mile of sewers	Extent of sewers considered
Alliance, Ohio	195,000	...
Altoona, Pa.	41,000	1.2 miles
Altoona, Pa.	86,000	0.6 miles
Altoona, Pa.	264,000	0.95 miles
Brockton, Mass.	45,000	2,000 ft.
Brockton, Mass.	61,000*a*	10,400 ft.
Brockton, Mass.	178,000*b*	10,400 ft.
Canton, Ohio	26,000	11 miles
Clinton, Mass.	32,500	...
Concord, Mass.	30,000	whole system
East Orange, N. J.	22,000*c*	29 miles
East Orange, N. J.	9,000	25 miles
Framingham, Mass.	35,000	whole system
Gardner, Mass.	45,000	whole system
Joint Trunk Sewer	25,000*d*	150 miles
Madison, Wis.	48,000	...
Malden, Mass.	50,000	whole system[1]
Marlboro, Mass.	50,000	whole system
Medfield, Mass.	25,000*e*	whole system
Metropolitan System	40,000*f*	137 miles
Natick, Mass.	80,000 to 100,000	8.58 miles
New Orleans, La.	32,000 to 60,000	...
North Brookfield, Mass.	24,000	1.41 miles
Peoria, Ill.	100,000	...
Reading, Pa.	5,000	...
Westboro, Mass.	1,072,000	3,010 ft.
Worcester, Mass.	32,000	...

a. Water in river low. *b*. Water in river high. *c*. Great precautions taken to prevent leakage, as construction was carried on in quicksand and the ground-water table was naturally 10 ft. or more above the sewer. *d*. This relates to the sewer serving parts of Newark and Elizabeth, N. J., and smaller places westward to Summit. *e*. Before house connections were made. *f*. Before any connections were made.

The Malden figures are from *Eng. News*, Aug. 27, 1903; the Concord figures from the 1900 report of the Sewer Commissioners, and the remainder from reports of the Mass. Board of Health and Trans. Am. Soc. C. E., vol. lxxvi, page 1909 *et seq.*

[1] 38 out of 45 miles total.

snow and ice and from gentle long-continued rains may readily percolate through the upper strata which later in the year form a hard compact crust more nearly impervious.

It is often held that sewers which at first are porous or have small cracks and poorly filled joints will gradually "silt up," that is, the pores will become filled with particles of fine clay and sand and that the leakage will thus be reduced. Trenches also become compacted and if in clay a nearly water-tight layer may be formed around the sewers, thus cutting off the water so that it will not follow along the pipes and enter through imperfect jonts. These observations are all more or less well founded, but it is also a fact, and this largely offsets the foregoing causes of reduced leakage, that many times the pipes crack after being laid and that connections made from time to time are so poorly constructed that they are the source of great leakage. Abandoned connections are rarely sealed at the sewers and may admit much water. Manholes are "heaved" by the frost so that water may enter between the courses. The net result of these changing conditions appears to be the presence of a gradually increasing quantity of ground water in the sewage.

As the water does not usually percolate or leak into sewers entirely around their perimeters, but rather enters near the water line, it seems hardly logical to report leakage in terms of area of masonry surface, of length of pipe joints or even of radius or diameter. It is doubtful even if the number of pipe joints per mile throws much light on the subject, although the chances of poor joints in the main sewer are proportional to the number of joints. This, however, takes no account of the leakage through house connections.

Data are most easily obtained in terms of quantity of leakage per mile of sewer and the most leakage may come from the smallest sewers. Having the data in this unit, it may for convenience be calculated in quantity per capita and quantity per acre, the latter being probably the most convenient form for use in planning intercepters and trunk sewers and in studies for pumping stations and treatment works. For detailed computations of small lateral sewers, the quantity per capita is perhaps most readily used.

The authors believe that an effort should be made to secure data in at least these three terms, gallons per mile of sewers, gallons per capita of population residing within the district served, and gallons per acre of this district.

ACTUAL MEASURED FLOW OF SEWAGE

In Table 56 are given statistics of the sewage of a number of Massachusetts cities and towns. These communities all have sewerage systems on the separate plan and the flows are consequently unaffected by

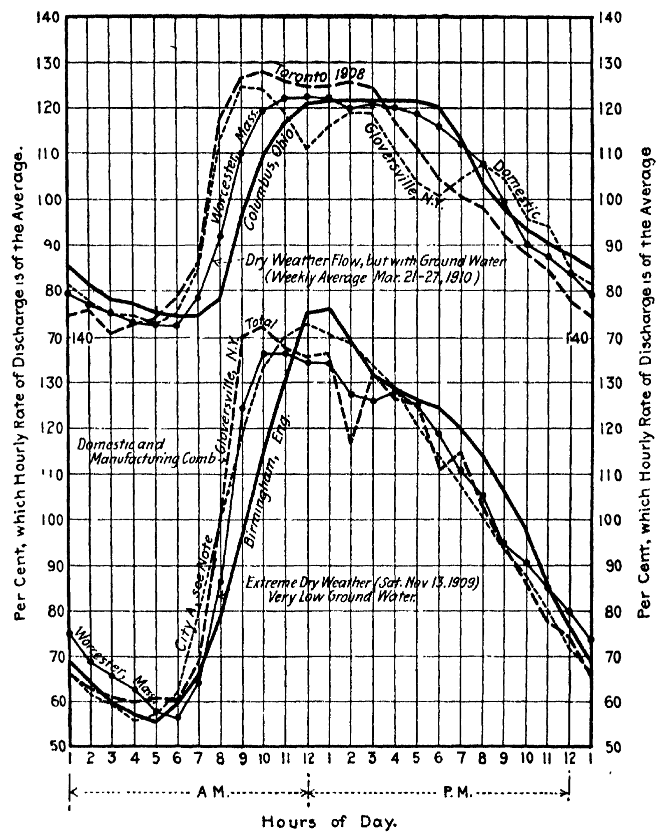


FIG. 63.—Hourly variations in flow of sewage at various places.

Worcester, Mass.—Estimated population, 150,000; average dry weather flow, 15,300,000 gal.; 1½ hours period of flow from city.

Toronto, Ont.—Dry weather flow, 73 gal. per capita; gagings made in city, October, November and December; from Report of City Eng., 1908.

Columbus, Ohio.—Discharge of intercepting sewer from weir measurements at the outfall, Dec 2-9, 1904; average week-day flow during extremely dry weather = 8,400,000 gal.; estimated population in 1905, 150,000; from Johnson's Report on Purification of Columbus Sewage.

City A (name must be omitted for local reasons).—From city to outlet is about 3 hours period of flow; population, 15,000, typical average dry-weather flow in July and August, 1908, 300,000 gal. per day.

Birmingham, Eng.—Gagings at treatment works; manufacturing wastes about one-fifth of dry-weather flow; two years, 1906-7.

Gloversville, N. Y.—Mill wastes about 26.2 per cent. of total flow; population, 20,000; average flow 2,000,000 gal. daily; gagings at experiment station, with half-hour period of flow from city; Oct. 30, 1906.

storm water except as it increases the leakage and the improper discharge of roof and surface water. It should be noted further that they are mostly flows from small communities without large quantities of trade wastes, and that the amounts per capita are much smaller than those to be expected in large cities.

Quite a different result is to be noted in certain sewer districts of Chicago, as indicated in Table 57. These sewers are on the combined plan, but the measurements were made during dry weather when the sewage presumably contained no storm water. The excessive flow in these sewers is to be accounted for largely by the great consumption of water in the city, which in 1910 averaged 242 gal. per capita of the population.

TABLE 56.—MAXIMUM AND AVERAGE FLOWS OF SEWAGE, 1903
(Massachusetts State Board of Health)

Place	Population	Average yearly quantity of sewage. Gallons per 24 hours				Average quantity of sewage in max. month. Gallons per 24 hours			
		Per inhabitant	Per person connected	Per connection	Per mi. of sewer	Per inhabitant	Per person conn	Per connection	Per mile of sewer
Andover	7,214	17	35	290	11,600	.	...	...	...
Brockton	44,202	20	35	512	26,930	31	55	799	41,990
Clinton	14,969	52	78	528	10,900	78	117	787	60,940
Concord	5,938	53	260	1,311	41,130	77	379	1,912	60,420
Framingham	12,376	53	87	537	41,400	78	129	796	61,400
Gardner sys	......	.	86	1,090	37,750	..	161	2,032	70,375
Gardner	11,792	47	......	...	...		...	...	...
Templeton sys	.....	.	56	714	33,780	..	..	..	...
Hopedale	2,513	60	75	750	37,500		..	...	...
Leicester	3,522	9	60	429	14,020	....		.	....
Marlborough	12,788	86	110	691	45,450	159	203	1,274	83,800
Natick	9,892	57	142	893	52,100	113	280	1,765	103,510
Pittsfield	22,549	65	97	797	45,930	69	104	854	49,240
Southbridge	11,090	32	159	1,108	61,400	...	..	......	.....
Spencer	7,635	49	125	625	37,500	..	...	......	......
Stockbridge	2,083	36	94	700	21,430	....	.....	....	......
Westborough	5,499	51	94	1,007	38,900	104	190	2,039	78,760

Variations in Flow.—The flow of sewage fluctuates between wide limits and follows somewhat the variations of the consumption of water. The day flow is also increased by the greater discharge of manufacturing wastes at that time. During the spring or wet months, the flow is increased by the added volume of ground water contributed, some of which is present at all times in most sewerage systems. In Fig. 63 are plotted the flows of sewage in terms of percentage of the average, from a number of cities. An attempt has been made to synchronize the curves by making allowance for the time required for the sewage to flow from the city

to the gaging point. The curves on the lower part of the figure are typical of dry weather conditions when ground water is at a minimum, while the curves on the upper portion of the figure are typical of conditions when ground water is relatively high.

TABLE 57.—TYPICAL DRAINAGE AREAS AND DRY WEATHER RUN-OFFS, CHICAGO, 1910 AND 1911

(From Wisner's Report on Sewage Disposal San. Dist. of Chicago, 1911)

Sewer outfalls	Drainage area in acres	Population estimated 1911	Dry weather run-offs, Cu. ft per sec.	Dry weather run-offs, Cu ft. per sec. per acre	Dry weather run-offs, Cu ft per sec, per sq mile	Dry weather run-offs, Gal per cap. per 24 hours	Density of population per acre	Period covered by observation
Diversey Boulevard (W).	890	23,550	8.65	0 0097	6.22	238	26 4	Aug 15-17, 1911, 2 days.
Randolph St (W)	240	11,368	6 10	0.0254	16 25	348	47 4	Aug, 3 days.
Robey St. (S) . .	2,500	38,728	10 1	0.0040	2 58	169	15 5	June 1-3, 1911, 2 days.
Ashland Ave, (S)	980	44,581	23 2[1]	0 0237	15 1	338	45 5	May 18-20, 1911, 2 days.
Center Ave., (S) .	660	23,463	20 9[2]	0 0317	20 3	578	35 6	May 16-18, 1911, 2 days.
Thirty-ninth St., pumping station.	14,340	285,900	140 0[3] 100.0[4]	0.0098 0.0070	6 25 4.46	318 227	20 0	. . . .
Ninety-second St .	98	3,666	1.84	0.0188	12.0	325	37.4	209 days. Aug. 1, 1910 Mar. 31, 1910
Wentworth Ave., (S), (Calumet)	5,300	30,464	12.4	0.0023	1.5[5]	264	5 8	Aug. 1, 1910. July 31, 1911 253 days

[1] Daily variation average
8 A.M. to 8 P.M. 28.5 c f p.s. contains large amount of industrial waste.
8 P.M. to 8 A.M. 18 6 c.f p.s.

[2] Daily variation average
8 A.M. to 8 P.M. 25.7 c.f.p.s. contains large amount of industrial waste.
8 P.M. to 8 A M. 17.0 c f p.s.

[3] This run-off or more for 76 days in 1909.

[4] This run-off or more for 276 days in 1909.

[5] 2.4 c.f.p.s. per square mile occurred 329 days in the year.

Two curves are shown in Fig. 64, taken from the report of the Sewage Disposal Commission of Milwaukee, 1910. The dotted line represents the flow from a large residential sewerage district in Milwaukee. The smooth curve is drawn through points obtained by averaging points taken from several curves representing the flow from the cities named in the note accompanying the illustration. In this case the curves were synchronized and an effort made to produce a curve typical of the fluctuations in flow of the sewage from the larger cities.

Obviously the fluctuations will be greater in single lines of sewers, or in small districts, than in trunk and intercepting sewers serving large areas.

The fact that the sewage requires a longer time to flow from certain districts than from others assists in producing a more nearly uniform flow in the intercepting sewers, as is evident from Fig. 65.

The flow on different days of the week varies considerably. On Sunday the quantity is smaller, and on Monday larger, than on other

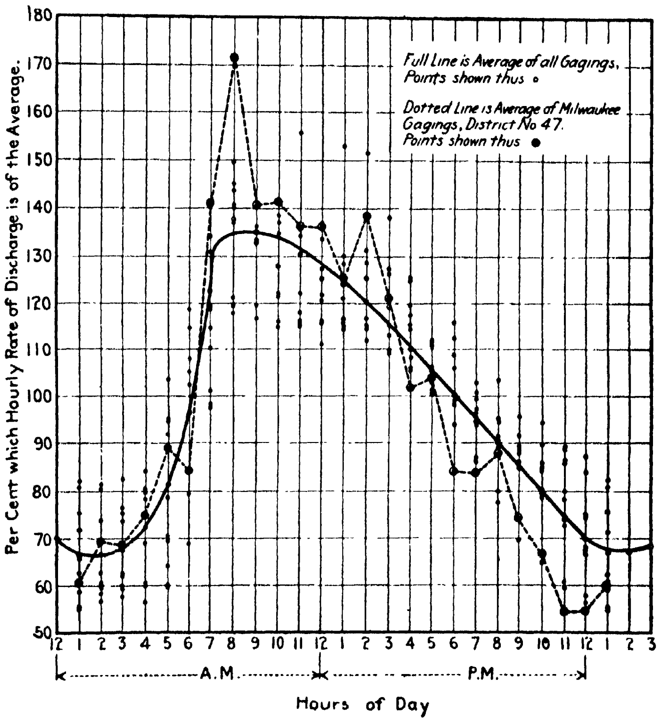


FIG. 64.—Hourly variation in flow of sewage in various cities.

The records used in preparing this diagram were from the following sources: Birmingham, England, average of two years, 1906–7; East Orange, N J., March 16–17, 1910; Gloversville, N. Y., Oct. 30, 1906, and Sept 12, 1907; City A, of 15,000 population, typical average curve; Milwaukee, Wis, Oct 24-28, 1910; Toronto, Ont., 1900 and 1908; Worcester, Mass., Nov. 13, 1909, and March 21-27, 1910.

days. On Monday the rate of the maximum flow is usually somewhat higher than on other days. The typical curve of sewage flow for one week in a city of about 15,000 population is given in Fig. 66. This city will be termed city A in this discussion as the authors are not permitted to give its name.

The rate of infiltration of ground-water varies greatly from season to season, but does not usually fluctuate materially from hour to hour. As the proportion of ground water increases, the fluctuations in the total quantity of sewage flowing from hour to hour naturally decreases. This is illustrated by Fig. 67, showing typical curves of hourly flow of sewage at City A, with flows ranging from 360,000 to 1,180,000 gal. per day, the excess of the larger flows being due wholly to ground-water. From similar data the curve given in Fig. 68 has been prepared, illustrating a method by which it is possible to calculate the average rate of flow on any day when the flow at a given hour in the day is known. While this curve is not applicable to other cities, it illustrates a con-

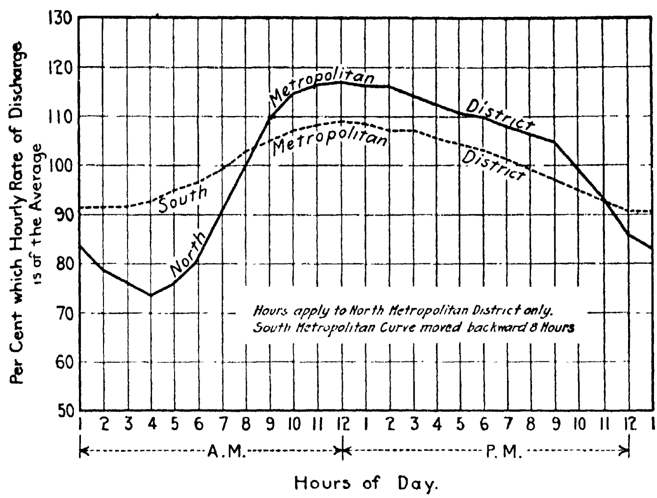


FIG. 65.—Hourly variation in flow of sewage in Massachusetts Metropolitan Districts.

venient method of obtaining fairly reliable records of the average quantity of sewage, by single daily observations. It is not as satisfactory as the use of a self-recording gage, and should not be employed where the latter is available.

RELATION OF TYPE OF DISTRICT TO QUANTITY OF SEWAGE

The quantity of sewage to be expected from a district depends upon its character. A residential district will produce sewage made up of the household wastes and the ground-water leakage, the former being governed by the quantity of water consumed, which will vary from 10

or less gal. per capita in the lowest class dwellings to 75 gal. in the first class dwellings or to 135 gal. in apartment houses, as shown by Table 47. A mercantile or commercial district will yield a much greater quantity on account of the great office buildings where water is used for many purposes, such as the operation of lavatories, motors and elevators. The flow from such districts will consist of the used water from the municipal supply, the ground-water infiltration and in many places the used water pumped from wells, which often amounts to large quantities. Manufacturing or industrial districts may contribute large quantities of liquid wastes. Some of this water is taken from the municipal supply but frequently very large quantities are taken from wells, rivers, lakes or even tidal bodies. The sewage from such districts is, therefore, made up of the used municipal supply from residences and industrial establishments, of the used private supplies of the manufactories, and of ground water. On the other hand districts comprising parks and cemeteries contribute only ground water, as a rule.

Classification of Areas.—A rational classification of areas in a city is a matter for careful study, due consideration being given to such natural conditions as topography and proximity to rivers, lakes or tidewater, and to such artificial conditions as railroad and street car lines, docks and canals. The residential districts usually occupy the uplands and sections topographically unsuited to industrial works. The commercial

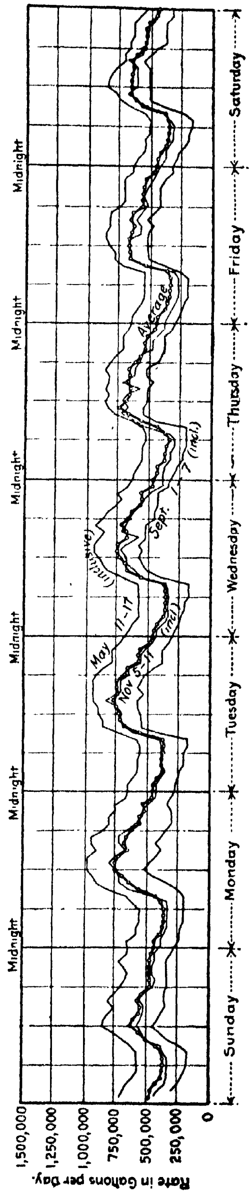


Fig. 66.—Fluctuations in quantity of sewage reaching treatment works of a city of 15,000 population.
The fluctuations indicated by the curves occur in the city from 2 to 3 hours earlier than at the works.

or mercantile districts occupy the more level areas in the "center" of the community, usually convenient to railroad terminals and docks, and contain public and office buildings, retail and wholesale stores, depots and freight houses, hotels, theaters, and generally some apartment houses. The commercial area is usually relatively small, and while provision should

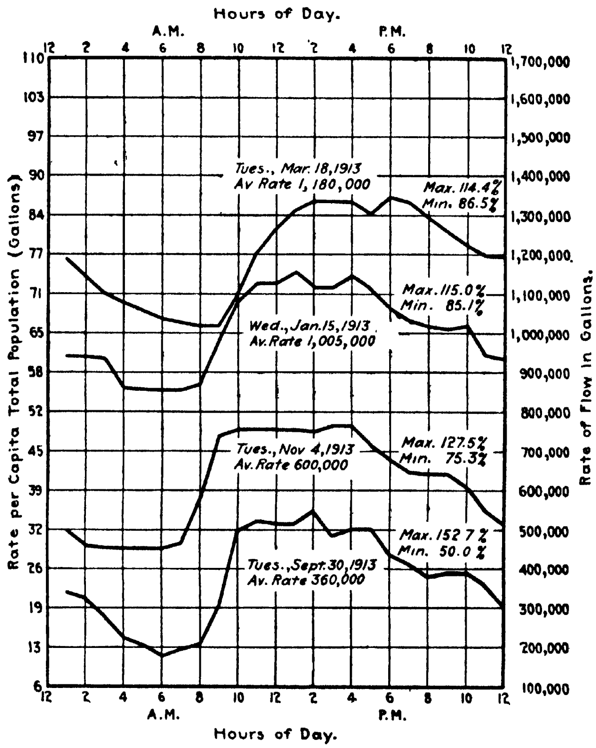


FIG. 67.—Hourly fluctuations in rate of flow of sewage containing different proportions of ground water.

be made for future growth care must be exercised to prevent estimating too large an area, for the unit quantities of sewage are large. Industrial areas are generally located on fairly level ground along the railroad lines, where spur tracks and sidings may be had. Works using large quantities of water are likely to be located along rivers or near docks where cheap water supplies may be had. These areas also are relatively small.

Philadelphia Sewer Gagings.—Gagings of the dry weather sewage flow from Philadelphia districts of different types of development were described in the annual report for 1912 by Mr. W. L. Stevenson. Some of the data procured are given in Table 58.

Residential Districts.—In computing the probable quantity of sewage from residential districts, it is first necessary to estimate the population likely to reside in them and decide upon the number of persons per acre for which provision should be made. In doing this it is not always safe to assume the same density as that estimated for the entire city, which rarely runs over 25 persons per acre, according to Table 41. The density

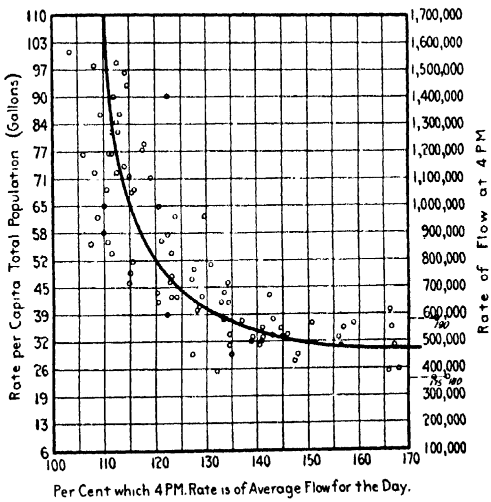


FIG. 68.—Ratio of flow of sewage at 4 P. M. to average flow per day in a city of 15,000 population.

in a particular district may run much higher; for example, one ward in Boston had a density of 190 in 1910, Table 42, and there are in most large cities smaller sewer districts in which the density of population greatly exceeds 25 per acre. The flow from the district is now readily calculated from the area, density of population, allowances for maximum rate of used water supply, and maximum rate of ground water.

In Cincinnati in 1912 the flow in a number of sewers serving residential, mercantile and industrial districts was gaged under the general direction of the authors by the Sewer Division of the Department of Public Service of that city, H. M. Waite, Chief Eng. The gagings were

made by E. J. Miner, assistant engineer, under the immediate direction of H. S. Morse, sewerage engineer.

TABLE 58.—SUMMARY OF DATA OBTAINED FROM GAGINGS OF DRY WEATHER SEWAGE FLOW, MADE IN 1910, PHILADELPHIA, PA.

		Area in acres		Population, census 1910		Average discharge per 24 hours, gallons	
Name of area	Character	Total	Settled 1910	Total	Per settled acre	Per settled acre	Per capita
Thomas Run	Residential, mostly pairs of two and three-story houses.	320	240	15,012	62.5	14,200	227
		426	337	21,677	64.0	9,860	153
		1,094	627	36,336	58.0	9,850	170
Pine St.	Residental, mostly solid four to six-story houses	160	156	15,152	97.0	26,300	271
Shunk Street	Residential, mostly rows of two and three-story houses.	208	208	25,751	123.0	10,500	85
		331	331	37,916	114.0	10,600	93
Lombard St.	Residential, tenements and hotels.	147	145	16,363	113.0	34,750	308
York St.	Residential and manufacturing.	358	351	33,340	94.0	36,000	383
		58	36	The population contributing sewage is not shown by the census figures		99,250	
Market Street	Commercial	123	80[1]			92,800	..

[1] This area is practically entirely built up. The settled area is "total" minus street area. None of the other areas is similar to it and the "settled area" includes the street area in each case.

Two districts consisting largely, but not exclusively, of residential property, were studied, the results being given in Table 59. Conventional curves were plotted as representing typical flows, the results being given in Table 60. In spite of the fact that these are large districts, it

TABLE 59.—AVERAGE FLOW OF SEWAGE FROM RESIDENTIAL DISTRICTS, CINCINNATI, OHIO, 1912

Sewer district	Area in acres	Population		Sewage flow from actual gagings: Gals. per acre per day		Gals. per capita per day		No. of gagings covering 24-hour day	Dates of gagings
		Total	Density	Avg.	Max.[1]	Avg.	Max.[1]		
Ross Run	1,617	17,912	11.1	1,028	2,820	93	254	2	Dec. 3, 4.
Mitchell Ave.	1,650	14,781	9.0	687	1,140	77	160	5	Nov. 19, 20, 21, 22, 23.
Totals and averages	3,267	32,693	..	857	2,130	85	207	....	....

[1] Maximum during gaging period.

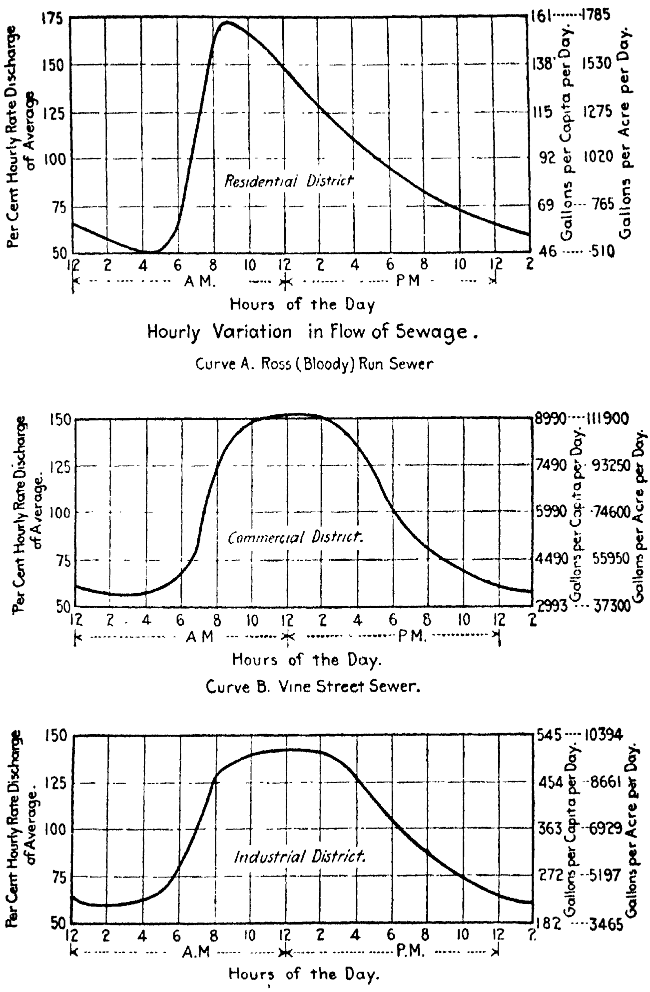


Fig. 69.—Hourly variations in flow of sewage, Cincinnati.

will be seen that the maximum rate of flow must be expected to reach from 160 to 170 per cent. of the average for 24 hours, and in the Ross Run district the maximum gaged flow was equivalent to 254 gal. per capita per day. A conventional or typical curve of flow is shown in Curve A, Fig. 69, and the rates of flow during the day are given for both residential and commercial districts in Table 60.

TABLE 60.—RATE OF SEWAGE FLOW FOR EACH HOUR OF THE DAY, IN PERCENTAGES OF THE AVERAGE RATE, CINCINNATI, OHIO

Time	Residential districts		Commercial districts								Industrial District
	Ross Run	Mitchell Ave.	Sycamore St.	Main St.	Walnut St	Vine St.	Race St.	Elm St.	Plum St.	Central Ave.	Marshall Ave.
1 A. M.	63	75	33	71	55	60	42	38	77	72	59
2	58	73	31	71	52	57	41	36	77	72	60
3	53	71	33	71	50	57	41	34	77	71	61
4	51	70	35	71	50	58	41	34	80	71	63
5	52	72	40	71	55	60	41	38	84	72	66
6	64	80	53	75	66	66	48	49	89	76	81
7	112	105	74	96	92	85	67	97	100	93	105
8	162	153	126	139	144	128	145	151	118	113	129
9	171	162	171	147	156	141	174	170	130	126	134
10	167	156	190	147	158	148	175	177	134	136	138
11	157	138	191	140	154	150	174	180	137	139	140
12 M.	148	123	190	135	150	152	173	178	137	139	141
1 P. M.	139	114	185	128	144	152	171	174	132	137	141
2	128	108	180	120	136	151	169	165	125	136	141
3	118	105	172	115	128	147	168	153	118	131	136
4	109	101	159	109	123	136	165	140	107	128	126
5	102	98	136	102	113	118	155	124	100	122	116
6	94	95	107	98	105	99	89	106	96	108	105
7	88	92	72	91	97	89	71	90	86	89	94
8	82	89	56	88	91	81	62	75	84	80	86
9	79	86	48	85	82	74	53	62	80	72	80
10	74	84	42	81	75	68	47	52	77	72	73
11	70	80	39	75	66	63	42	45	77	72	68
12 P. M.	66	77	36	72	61	60	42	40	77	72	63

These figures have been computed from the average or conventional curves for the several sewer districts. Sunday flow has not been included in preparing these conventional curves.

Mercantile Districts.—The allowance for used water from a mercantile district is more difficult to estimate than that from a residential district. If the estimate is to be made in connection with the design of interceptors, pumping stations or treatment works, in which cases the district sewers have usually been built, it is highly desirable to gage the flow in the

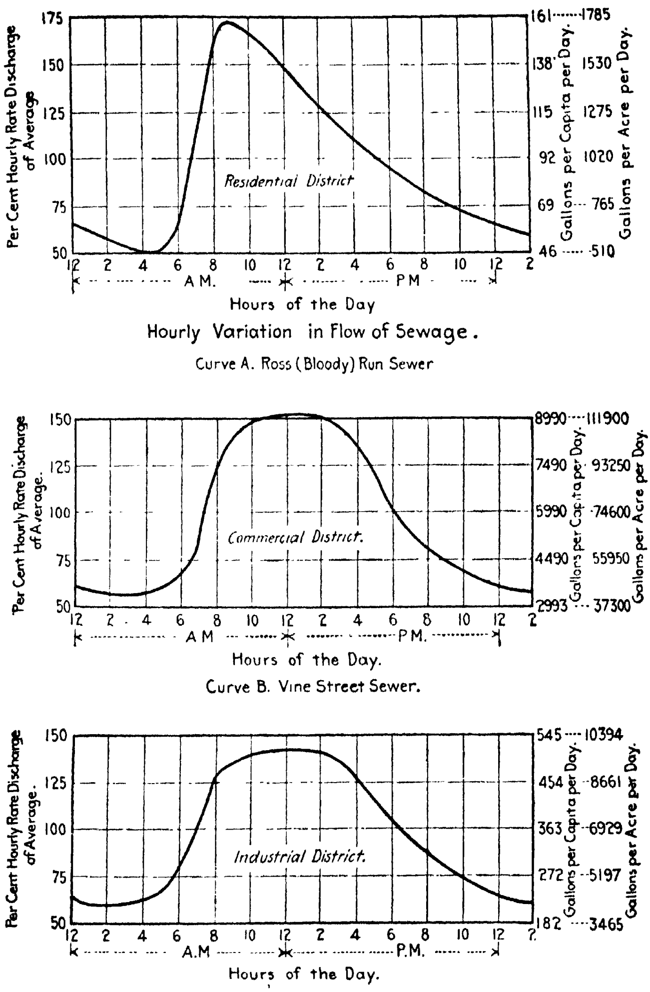


FIG. 69.—Hourly variations in flow of sewage, Cincinnati.

Industrial Wastes.—The amount of industrial wastes not originating in the public water supply is subject to wide variations in different cities and is a matter for individual study in any particular case. The amount of such wastes may be large in some cities and even exceed the volume of house sewage. The amounts of these wastes have been investigated or estimated in a number of cities and the results of a few of such studies are given in Table 62.

TABLE 62.—ESTIMATES OF INDUSTRIAL WASTES ENTERING SEWERS

City	Gal. per capita per day	Date of estimate
Milwaukee, Wis.	57	1911
Fitchburg, Mass.	81 (max.)	1911
Passaic Valley Sewer	38 (max.)	1908
Louisville, Ky.	57	1906
Paterson, N. J.	18 (max.)	1906
Providence, R. I.	42 (max.)	....
Mass. Neponset Valley interceptor.	25 (max.)	1895
Cincinnati, Ohio	50	1912

In Table 63 the results of gaging one industrial district are given. This district contained both residential and industrial areas but is typical of many sewer districts in industrial centers. These gagings extended over 3 days and the maximum rate of flow found was over 13,000 gal. per acre, equivalent to over 700 gal. per capita. The hourly fluctuations to be expected in this district, taken from a smooth curve based on the gagings, are given in Table 60 and the curve is shown as Curve C in Fig. 69.

TABLE 63.—AVERAGE FLOW OF SEWAGE FROM AN INDUSTRIAL DISTRICT, CINCINNATI, OHIO, 1912

Sewer district	Area in acres	Population		Sewage flow from actual gagings				No. of gagings covering 24-hour day	Dates of gagings
				Gals. per acre per day		Gals per capita per day			
		Total	Density	Avg.	Max.[1]	Avg.	Max[1]		
Marshall Ave.	294	5611	19.1	6787	13,485	356	708	3	Nov. 26, 27, 30

[1] Maximum during gaging period.

Estimate of Quantity of Sewage from Entire City.—Having given consideration to the population, area and average and maximum rates of flow to be expected in residential, mercantile, industrial and park districts, it is next necessary to combine the different elements to arrive at an estimate of quantity of sewage for which provision should be made for the entire community, or a portion of it which may be served by a trunk or intercepting sewer. It will simplify the explanation of the method of

TABLE 64.—POPULATION AND AREAS OF CINCINNATI SEWER DISTRICTS AS OF 1912 AND 1950 OHIO RIVER DISTRICT
(Part of table to illustrate form and method)

No.	Sewer districts. Name	As of 1912			As of 1950								
		Area inside city (acres)[1]	Popu-lation	Density persons per acre[1]	Area in acres							Density persons per acre[1]	Popu-lation
					Total area of sewer district	Parks and ceme-teries	Rail-road yards	Indus-trial area	Mer-cantile area	Residen-tial area	Total area excluding parks, cemeteries, R. R. yards		
50	Waldon St.	100.8	1,168	11 6	100.8	...	...	22.0	.....	78.8	100 8	15.0	1,510
51	Eggleston Ave.	1,509.8	50,435	39.5	1,509.8	203.0	29.2	271.8	161.0	844.8	1,277 6	49.3	63,000
52	Butler St.	12.3	400	32.5	12 3	....	. .	12.3	....	...	12.3	39.8	490
53	Pike St.	10.5	446	42.5	10.5		.	10.5	..	.. .	10.5	40.0	420
54	Lawrence St.	15.1	368	24.4	15.1	.	.....	15.1	... .	. . .	15.1	39.7	600
55	Ludlow St.	14.4	928	64.5	14.4	.	..	14.4	. . .	. .	14.4	65.3	940
56	Broadway.	26.0	1,959	75.3	26.0	...	.	12.0	14.0	. ...	26.0	80 0	2,080
57	Sycamore St.	37.9	1,702	44.9	37.9	.	.....	8.0	29.9	.....	37 9	60 0	2,270
58	Main St.	28.4	540	19.0	28 4	.	.....	5.0	23.4	......	28 4	25.0	710
59	Walnut St.	36.9	421	11.4	36 9	...	. ...	6.0	30.9	.. ....	36.9	10 0	370
60	Vine St.	33.7	308	9.1	33.7	... .	.	7.0	26.7	....	33.7	10 1	340
	Totals of districts 40 to 80 incl.	15,614.3	238,794	16.0	17,266.5	540.2	118.0	2,092.0	721.2	13,795.1	16,608.3	18.3	303,826

[1] In computing density of population, areas of parks, cemeteries, railroad yards are deducted from total area of sewer district.

TABLE 65.—ESTIMATED QUANTITY OF CINCINNATI SEWAGE TO BE PROVIDED FOR AT MAXIMUM RATE OF FLOW AS OF 1950
OHIO RIVER DISTRICT
(Part of table to illustrate form and method)

No.	Sewer districts name	Estimated future population	Area in acres: Total area	Area in acres: Industrial area	Area in acres: Mercantile area	Million gallons per day: Water supply reaching sewers 135 g.c.d.	Million gallons per day: Ground water 750 g.a.d.	Million gallons per day: Industrial sewage 9000 g.a.d.	Million gallons per day: Mercantile sewage 40,000 g.a.d.	Estimated total quantity: m.g.d.	Estimated total quantity: c.f.s.	Cumulative quantities: m.g.d.	Cumulative quantities: c.f.s.
50	Waldon St.	1,510	100.8	22.0	.	0.204	0.076	0.198	.	0.478	0.7	14.061	21.7
51	Eggleston Ave......	63,000	1,509.8	271.8	161.0	8.505	1.132	2.446	6.440	18.523	28.7	32.584	50.4
52	Butler St.	490	12.3	12.3	. .	0.066	0.009	0.111	.....	0.186	0.3	32.770	50.7
53	Pike St............	420	10.5	10.5	....	0.057	0.008	0.094	.... .	0.159	0.2	32.929	51.0
54	Lawrence St.....	600	15.1	15.1	....	0.081	0.011	0.136	.	0.228	0.3	33.157	51.3
55	Ludlow St.........	940	14.4	14.4	....	0.127	0.011	0.130	..	0.268	0.4	33.425	51.7
56	Broadway	2,080	26.0	12.0	14.0	0.281	0.020	0.108	0.560	0.969	1.5	34.394	53.2
57	Sycamore St. .	2,270	37.9	8.0	29.9	0.306	0.028	0.072	1.196	1.602	2.5	35.996	55.7
58	Main St..	710	28.4	5.0	23.4	0.096	0.021	0.045	0.936	1.098	1.7	37.094	57.4
59	Walnut St.........	370	36.9	6.0	30.9	0.050	0.028	0.054	1.236	1.368	2.1	38.462	59.5
60	Vine St..	340	33.7	7.0	26.7	0.046	0.025	0.063	1.068	1.202	1.9	39.664	61.4
	Totals of districts 40 to 80 inclusive...	303,826	17,266.5	2092.0	721.2	41.017	12.950	18.828	28.848	101.643	157.2	..	.

Note.—In this table, g.c.d is an abbreviation of gallons per capita daily: g.a.d. is an abbreviation of gallons per acre daily: m.g.d. stands for million gallons daily, and c.f.s. for cubic feet per second.

estimating and serve to summarize the whole discussion, if an illustration from actual practice is given. For this purpose the studies made in Cincinnati, Ohio, in 1913, already alluded to, may be taken. Under the conditions three main intercepter districts were decided upon and designated as the Duck Creek, Ohio River and Mill Creek districts from the names of the water-courses along which the intercepters are to be constructed.

Having first studied the local conditions and estimated the probable growth of the city as a whole, both in population and area, during the assumed "economic period of design," consideration was given to the distribution of population and area among the several sewer districts. The respective areas, as dictated by topography, were indicated upon maps and measured with the planimeter. A large map was then prepared upon which were indicated the outlines of the residential, mercantile and industrial areas and parks, railroad yards and cemeteries. The portions of each coming within each sewer district were measured and tabulated as in Table 64, together with the estimated future population.

Consideration was next given to the quantity of sewage for which provision should be made, the units of maximum rate of flow in intercepters adopted after a study of local conditions and all data available being given in Table 66.

TABLE 66.—UNIT QUANITIES OF FLOW IN INTERCEPTERS ASSUMED FOR CINCINNATI, OHIO

Residential areas:	
Sewage	135 gal. per capita per day
Ground water	750 gal. per acre per day
Mercantile areas:	
Sewage (resident population)	135 gal. per capita per day
Additional allowance for character of development.	40,000 gal. per acre per day
Ground water	750 gal. per acre per day
Industrial areas:	
Sewage (resident population)	135 gal. per capita per day
Industrial wastes	9,000 gal. per acre per day
Ground water	750 gal. per acre per day
Parks, railroad yards and cemeteries:	
Ground water	750 gal. per acre per day

The results obtained by the computations are illustrated by Table 65 and are summarized for the whole city in Table 67.

A few comments may serve to explain some of the reasons for the units adopted. The rates in all cases are the highest anticipated at times when it will be necessary to intercept the sewage or ultimately to treat it. They are also influenced by the smoothing out effect of the differences in time of entrance of sewage from lateral sewers into trunk sewers

TABLE 67.—ESTIMATED TOTAL QUANTITY OF SEWAGE TO BE PROVIDED FOR AT MAXIMUM RATE OF FLOW IN THREE INTERCEPTER DISTRICTS, CINCINNATI, OHIO, AS OF 1950

Drainage district	Estimated future population	Area in acres: Total area	Industrial area	Commercial area	Million gallons per day: Water supply reaching sewers 135 g.c.d.	Ground water 750 g.a.d.	Industrial sewage 9000 g.a.d.	Commercial sewage 40,000 g.a.d.	Estimated total quantity: m.g.d.	c.f.s.	Quantity in units of: Gallons per acre per day	Gallons per capita per day
Mill Creek...	308,664	52,740.1	4,863.3	....	41.670	39.555	43.770	....	124.995	193.4	2,370	405
Ohio River....	303,826	17,266.5	2,092.0	721.2	41.017	12.950	18.828	28.848	101.643	157.2	5,885	334
Duck Creek...	99,320	12,690.9	1,228.5	....	13.408	9.518	11.056	....	33.982	52.6	2,680	342
Total......	711,810	82,697.5	8,183.8	721.2	96.095	62.023	73.654	28.848	260.620	403.2	3,150	366

and from trunk sewers into intercepters. The ground-water allowance is very low, because of the enormous area and sparse population anticipated, and because of the topography, which assures the construction of a large portion of the sewerage system above the water table during the drier portions of the year. These conditions appeared to warrant the adoption of a ground-water factor much lower than the authors have dared to use in a number of other cities.

The proportions of the several classifications of flow in the several districts and for the entire city are given in Table 68. From these data it will be seen that there are great differences in the allowances for the several intercepters, more than twice as great a flow per acre being provided for in the Ohio River intercepter as in either of the others.

Provision for Storm Water.—There is a general impression that it is wise to provide in intercepting sewers for a small quantity of storm water, expressed often as being sufficient for the "first flushings" of street surfaces and sewers. This impression is based upon the assumption that there are accumulations of sewage sludge in the sewers and quantities of filth on the streets which will be immediately flushed into the intercepting sewers with the first run-off due to rain. In some sewers laid on very flat grades and where sewers have settled or have been built with depressions in them, there may be such deposits, but where sewers are laid on grades which give satisfactory velocities such deposits

are believed to be exceptional. Where deposits occur they are generally found to consist largely of sand and other heavy detritus which will be carried along only by relatively high velocities.

TABLE 68.—ESTIMATED UNIT QUANTITIES OF SEWAGE TO BE PROVIDED FOR AT A MAXIMUM RATE OF FLOW IN THREE MAIN DRAINAGE DISTRICTS, AS OF 1950, CINCINNATI, OHIO.

	Duck Creek interceptor			Ohio River interceptor			Mill Creek interceptor			Whole city		
	Gal per acre per day[1]	Gal per capita per day	Per cent	Gal per acre per day[1]	Gal. per capita per day	Per cent	Gal. per acre per day[1]	Gal per capita per day	Per cent	Gal per acre per day[1]	Gal per capita per day	Per cent
Sewage from resident population	1057	135.0	39.5	2375	135.0	40.1	790	135	33.4	1162	135.0	36.9
Additional allowance, mercantile areas				1670	95.0	28.1		.	.	319	40.6	11.1
Additional allowance industrial areas	873	111.2	32.5	1090	62.0	18.5	830	142	35.0	890	103.3	28.2
Ground water	750	95.8	28.0	750	42.0	12.7	750	128	31.6	750	87.1	23.8
Totals	2680	342.0	100.0	5885	334.0	100.0	2370	405	100.0	3150	366.0	100.0

[1] Total area of district.

NOTE.—Industrial sewage, based upon 9000 gal. per acre of industrial area; mercantile sewage, upon 40,000 gal. per acre of mercantile area; ground water, upon 750 gal per acre of total area; domestic sewage, upon 135 gal. per capita.

Interceptors are fed by trunk sewers serving rather large districts. Considerable time is required to flush the major part of the systems to the interceptor, during which a large flow is likely to reach them from the nearer portions. Unless considerable surplus capacity is provided, the interceptors will often be running full before the flushings from much of the tributary area can reach them. Therefore, too much stress should not be laid on their ability to care for the "first flushings," although as ordinarily designed they can accomplish something in this direction.

Assuming that the average flow of sewage is 100 gal. per capita and that the capacity of the interceptor is 300 gal. per capita, there will be a surplus capacity available for "first flushings" equivalent to twice the average flow of sewage, if such flushings come at a time when the flow is at the average rate. The maximum rates of flow generally occur in the spring when ground water is high and at other times there will always be some surplus capacity. Furthermore, as interceptors are built for many years, there will be a considerable excess capacity during the earlier years, although this should not usually be counted upon to care for storm water, for it is a gradually diminishing allowance accompanied by a gradually increasing need, if need there should be.

Under the foregoing conditions the sewage will be diluted to three times its normal flow. Furthermore, consideration must be given to the excess of water used in this country and to the quantity of ground water which leaks into the sewers. With these eliminated the quantity of sewage would be comparable with that obtained in Europe. Taking all these conditions into account, it is evident that the dilution approaches the standard of the Royal Commission on the Disposal of Sewage, which is six times the dry-weather flow.

In view of these conditions, it was not deemed wise to provide capacity in the Cincinnati intercepters for storm water in excess of that which can be carried when the flow of sewage is less than the maximum rates assumed as already described, in other words no special allowance for storm water was made.

Caution.—The foregoing outline of a rational method of estimating the quantity of sewage to be provided for applies to the design of intercepters and large trunk sewers and the units adopted are for maximum rates of flow when the sewers are running full.

AVERAGE RATES OF FLOW

The average rates of flow upon which estimates of cost of pumping and treatment may be based are much below the maximum rates, and, from the data available, appear to range in a general way between 100 and 125 gal. per capita per day for the larger cities. For small towns average rates appear to range from about 25 to 60 gal. per capita per day.

Bases of Design of Existing Intercepters.—The allowances made by engineers in the design of a number of existing intercepting sewers are given in Table 54. Some of the older intercepters, designed on a basis of less than 300 gal. per capita, now appear to be inadequate for the service they will ultimately be required to render and more recent designs are more liberal.

CHAPTER VI

PRECIPITATION

The sizes of combined sewers and storm-water drains are determined primarily by the rates of rainfall and the available slopes for the sewers. The study of precipitation and the run-off from areas of different shapes, slopes and surface characteristics should never be neglected by the engineer interested in sewerage works. Until about 1910 there was a general tendency among engineers to rely on various formulas for run-off, but about that time the belief began to spread through municipal engineering offices and among consulting specialists that there was great need of more complete and more accurate knowledge of rainfall and run-off, upon which to base the calculation of sewer sizes.

Most precipitation records give only the total amount of rainfall day by day, or at most the total precipitation during each storm, together with the times of beginning and ending. Such records are of slight value in the study of storm-water run-off. It is the maximum rate of precipitation lasting for a sufficient time to produce maximum run-off conditions, which is of importance. Rates of precipitation can only be obtained from the records of automatic recording rain-gages. The use of such gages has generally been limited to the larger cities, including the more important Weather Bureau Stations and engineering offices where run-off problems have been studied in detail, and until within recent years very little trustworthy information relating to maximum rates of precipitation has been available.

AUTOMATIC RAIN GAGES

The principal types of automatic rain-gages are the following:

The Fergusson Gage.—This instrument, Fig. 70, is made by the International Instrument Co., Cambridge, Mass., and costs about $80. The total height is 32 in. and the outside diameter 18 in., the diameter of the collector ring being 8 in. It has a total capacity of 6 in. of rain. The water received by the collector is discharged into a can supported upon a spring balance, the movement of which is transmitted by link motion to a pen moving through an arc of a circle and making a record upon a chart carried by a revolving drum. The length of the chart is 11 in., representing 24 hours of time, and accordingly the time scale is small, Fig. 71. The height of the chart is 6 in. and the precipitation is recorded to natural scale.

The advantages of this instrument are its comparatively small size and the fact that it is entirely self-contained and is not dependent upon the operation of electrical recording apparatus. The disadvantages are the

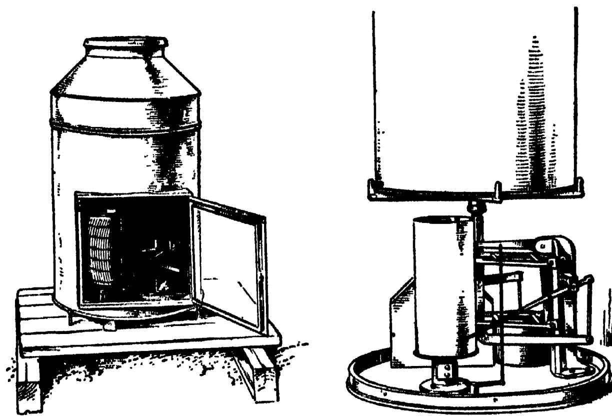
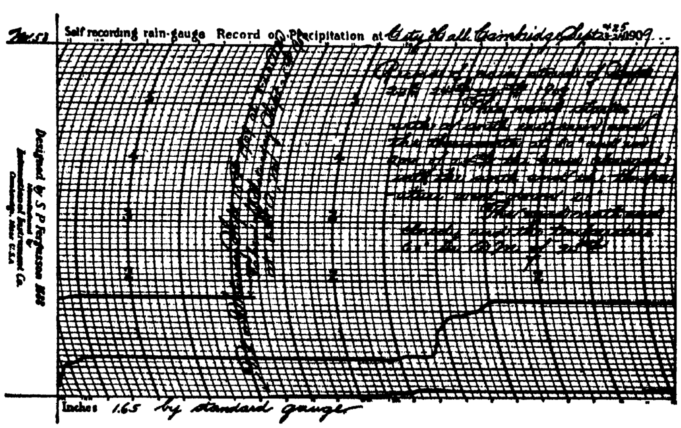


FIG. 70.—Fergusson rain gage.



Actual time scale is ½ inch to 1 hr.

FIG. 71.—Chart from Fergusson rain gage.

very small time-scale, making it impossible to determine accurately high rates of precipitation, and the further fact that the pen moves in a curved line, rendering the chart somewhat difficult to interpret. Moreover, the

link motion contains several joints and there is possibility of lost motion and friction in the joints.

Draper Gage.—The Draper Manufacturing Co., New York City, makes the recording rain-gage shown in Fig. 72. This gage is 26 × 15 × 10 in. in size, weighs 45 lb. and sells for $75. It is in general similar to the Fergusson gage. The principal differences are that the gage has a capacity of but 5 in., instead of 6; the motion of the pen is from the top of the chart down instead of from the bottom up, and the time scale is slightly greater (if the clock is adjusted to revolve the drum once each day). The circumference of the drum is about 12 1/2 in. so that 1 hour of time corresponds to a little more than 1/2 in. on the chart. Substantially the same comments apply to this gage as to the Fergusson gage, except those relating to the link motion. In the Draper gage, the arm carrying the pen is actuated by a fine wire passing over a quadrant instead of by a link motion.

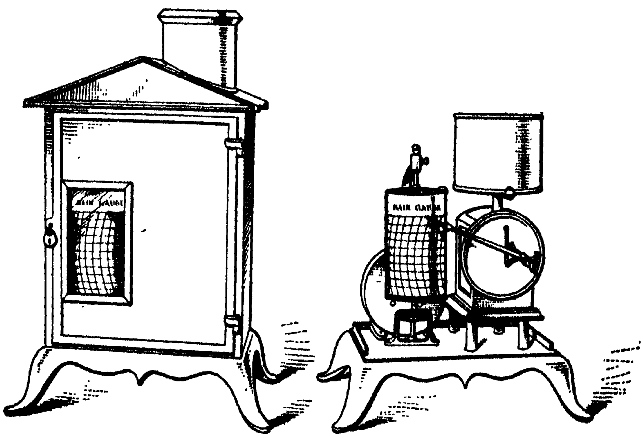


FIG. 72.—Draper rain gage.

Draper Gage, Old Pattern.—In the old pattern of the Draper gage the water was collected through a funnel placed in the roof of the building or chamber and conveyed through a tube into a tipping bucket suspended from two helicoidal springs, Fig. 73. These springs were so adjusted that the scale of precipitation was magnified ten times, that is to say, 1 in. on the chart represented 0.1 in. of rain. The pen arm was attached to the bucket and moved directly with it, from the top of the chart to the bottom. When 0.5 in. of rain had been collected the bucket dumped and immediately returned to its upright position bringing the pen to zero at the top of the chart.

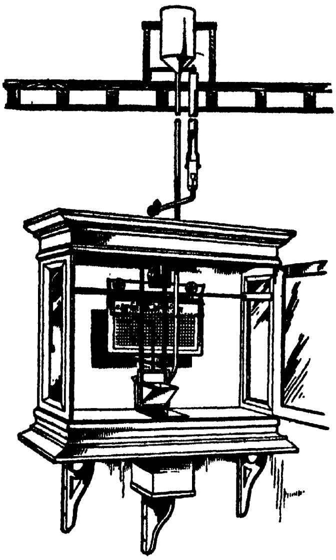
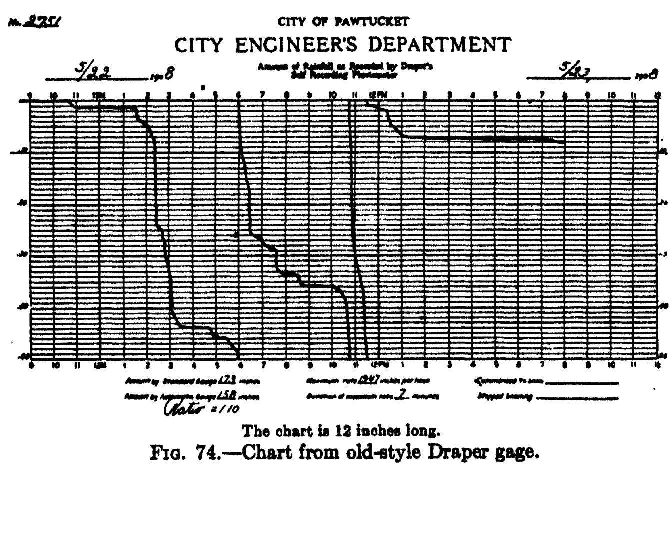


FIG. 73.—Draper rain gage, old style.

The chart, Fig. 74, was carried on a flat plate suspended from a track and moved by clockwork. As originally constructed the chart was made to represent 1 week of time, but more recently the clockwork has been modified so that the chart makes a complete traverse in 24 hours. It is, however, so short, the total length being 12 in., that the scale is very small, only 1/2 in. per hour. The price of it was $175.

By this instrument the precipitation is unnecessarily magnified, while the time-scale, as in the case of most instruments on the market, is too small for accurate determination of high rates of rainfall. The necessity of placing the collector upon the roof of the building, or else constructing a vault for the recorder, is also objectionable.

A recent improvement of the old type of Draper gage, devised by George A. Carpenter, city engineer of Pawtucket, R. I., makes possible



The chart is 12 inches long.

FIG. 74.—Chart from old-style Draper gage.

the exact determination of 2-minute intervals of time, and accordingly the maximum rates of precipitation for very short periods can be accurately determined. This result is accomplished by tapping the pen at intervals of 2 minutes in such a way as to make a dot heavier than the line traced by the pen and therefore readily distinguishable. Thus the amount of precipitation in each 2 minutes can be read from the chart with great accuracy.

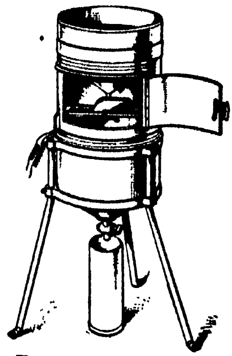


FIG. 75.—Friez tipping bucket gage.

Friez Gage.—This is one of the most widely used automatic rain-gages. It is made by Julien P. Friez, Baltimore, Md. The price of the instrument is $53.75 and of the recorder $65., making a total of $118.75. In this instrument, Fig. 75, rain is collected in a funnel 12 in. in diameter and conducted through a tube into a bucket containing two compartments. The contents of each compartment are equivalent to 0.01 in. of rain. The bucket is supported on trunnions in such a manner that as soon as a compartment is filled it tips and discharges the ac-

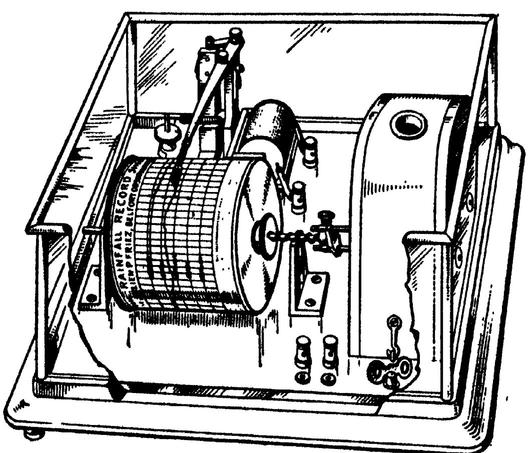
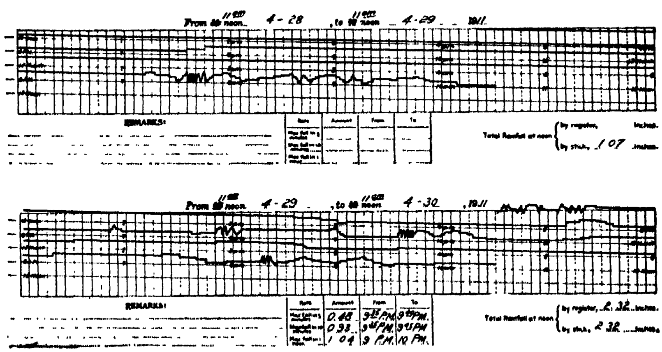


FIG. 76.—Register for Friez gage.

cumulated rain, presenting the other compartment for filling. Each time the bucket tips it makes an electrical contact and causes a pen to

record a step upon a chart carried by a revolving cylinder. A sample chart from this instrument is shown in Fig. 77. It is seen that the curve traced does not represent directly the progress of the storm, the motion of the pen being reciprocating, up for 0.05 in. and down for 0.05 in.

The time-scale of this chart is 2-1/2 in. to an hour. The amount of rainfall is indicated, not by measurement on the chart, but by counting the number of steps, or of "flights" of 10 steps each. It is therefore possible to determine the rates of rainfall from this record with a very good degree of precision. The possibilities of error are, however, considerable. The Weather Bureau carefully investigated the accuracy of the instrument and determined that on account of the appreciable time required for the bucket to tip, the error due to inflow of water into a compartment already full before the bucket could tip and present the



The chart is 10½ inches long.
FIG. 77.—Chart from Friez gage.

empty compartment is sufficient to produce an error of about 5 per cent. at times of very heavy rain. It is found, also, that dirt washed into the buckets from the dust accumulating in the gage, affects the character of the surfaces and the accuracy of the record. The adjustment of the instrument must be carefully made and its record is absolutely dependent upon the electrical apparatus working correctly.

A test of such a gage, made by J. B. F. Breed, Chief Engineer of the Commissioners of Sewerage of Louisville, Ky., showed a total discrepancy of 17 per cent. in a rainfall amounting to a total of 1.70 in. with a maximum intensity of 7.68 in. per hour for 5 minutes and an average intensity of 1.65 in. for 60 minutes. By discharging water into the gage at various rates and measuring the actual accumulation as compared with the recorded collection, it was found that the rates of precipitation computed from the gage record should be increased by about 2 per cent. for each

inch per hour. Thus a record showing precipitation at the rate of 5 in. per hour should be corrected by adding 10 per cent., making the corrected rate 5.50 in. per hour. The test was carried to an observed rate of 8.40 in. per hour, the actual rate being 9.78 in. per hour. It has also been found that the bucket sometimes stops on center, thus failing to register entirely, as a portion of the water flows out each side and the bucket no longer tips.

Queen Gage.—This instrument, made by Queen-Gray Co., Philadelphia, Pa., is of the same pattern as the Friez tipping bucket gage. It

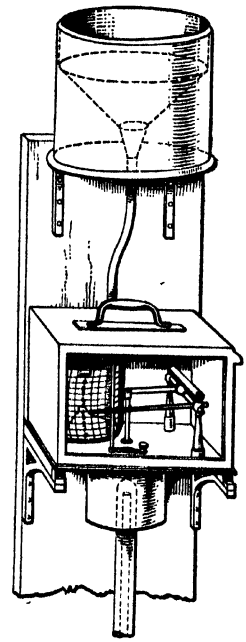


FIG. 78.—Richard "A" gage.

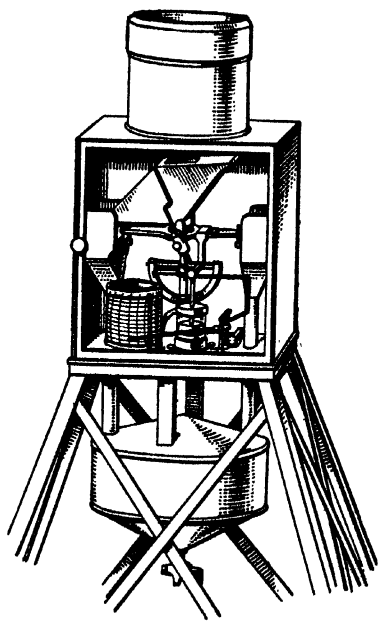


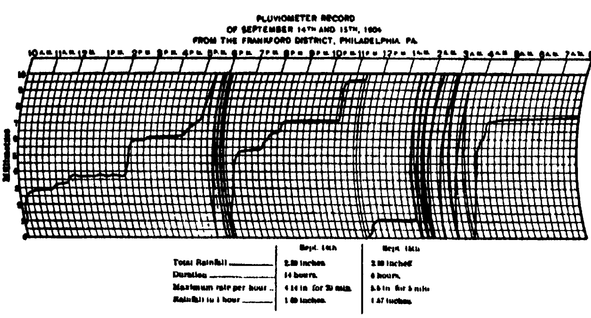
FIG. 79.—Richard "B" gage.

is sold complete, with registering device, for $100. The recorder is somewhat smaller than that of the Freiz gage so that the time-scale is 2 in. per hour instead of 2-1/2 in. as in the Friez gage, and the movement of the pen for each 0.01 in. of rain is also somewhat less.

Richard Gage.—This instrument is made by Jules Richard of Paris and is sold in the United States by Ernest H. Du Vivier, New York City. It is made in two patterns, as shown by Figs. 78 and 79. Model A has a collector 8.3 in. in diameter, the total height of the instrument

being 57 in. and its width 10.6 in. The selling price in the United States is $78. The rain is accumulated in a reservoir connected through a weighing device with a pen recording upon a chart carried by a revolving cylinder. The height of the chart is 2.9 in., which represents 0.4 in. of rain. When this amount has accumulated the reservoir is emptied by a siphon started by an electrical apparatus, and the pen returns to zero ready to record the next filling of the reservoir. The length of the chart is 12 in. which may be made to represent either 1 day or 1 week. The latter graduation is absolutely useless for recording maximum rates of rainfall, and even with the former the time scale would be but 1/2 in. per hour so that it would be impossible to measure short times with any degree of accuracy.

This instrument is open to the objection that the period during which the receiver is emptying may be considerable and thus introduce a



The chart is 12 inches long.

FIG. 80.—Chart from a Richard gage.

material error. Moreover, the time-scale is very short and the pen moves in a curved line, both of which are objectionable features.

Model B of the Richard gage contains a tipping bucket so designed that it tips gradually with increasing accumulations of water, and does not dump until 0.4 in. of rain has accumulated. The motion of the bucket is transmitted through a link to a pen marking, as in Model A, upon a chart of the same kind. This instrument has a collector 8.3 in. in diameter, and the total height of the instrument is 61 in. The maximum width is 13.8 in. Its selling price is $171. The chart is the same as in Model A, and the time-scale is too small for accurate determination of the rate of precipitation. The motion of the pen being in a curved line is also objectionable, and the possibility of errors in the transmission of the movement of the bucket to the pen are considerable. It is also subject in some degree to the same possibilities of error as the Friez gage.

A sample record from a Richard gage, used in Philadelphia, is shown in Fig. 80.

Marvin Gage.—This is a very elaborate gage of the weighing type,

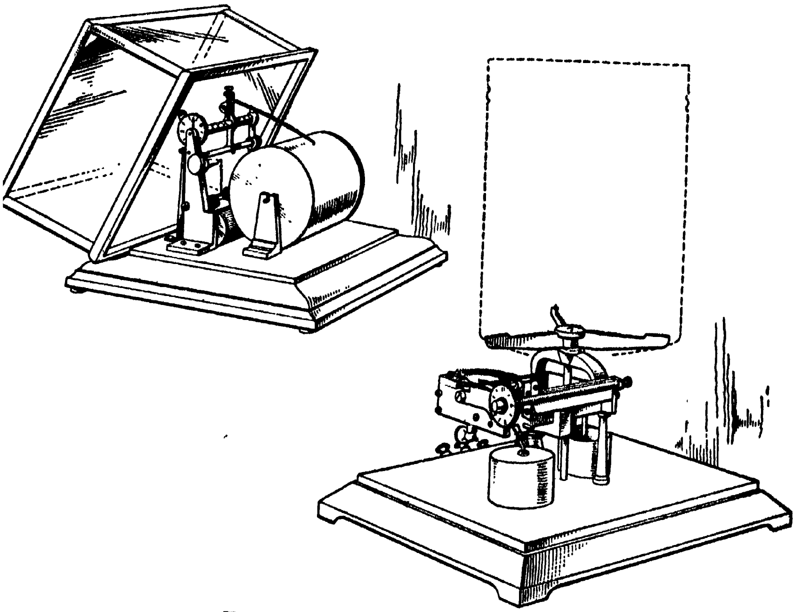
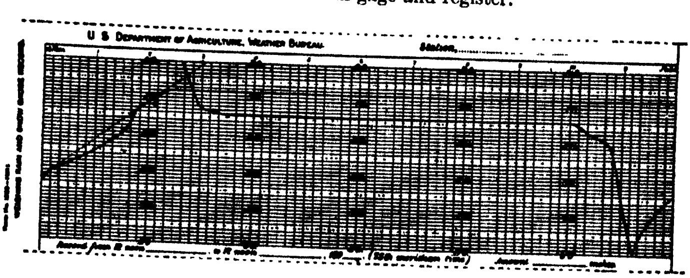


FIG. 81.—Marvin gage and register.



The chart is 8½ inches long.

FIG. 82.—Chart from a Marvin gage.

devised by Prof. C. F. Marvin of the U. S. Weather Bureau, and constructed by Julien P. Friez of Baltimore. It is used at only a few of the most important Weather Bureau stations. It is described in detail in

Circular E, Instrument Room, U. S. Weather Bureau. The precipitation is received and retained in a pan supported on a scale beam, Fig. 81. Deflection of this beam makes an electrical contact, causing a record, and also a movement of the counterweight to again balance the beam. This record is made for each 0.001 in. of rain. The pen moves back and forth across the record sheet, which is nearly 3-1/2 in. wide, the entire motion in one direction corresponding to 1 in. of rain, so that the depth collected is magnified nearly 7 times, Fig. 82. The time-scale, however, is comparatively small, only 1 in. per hour, but intervals of time as short as one minute can be determined, so that it is possible to determine rates of precipitation with a very good degree of accuracy.

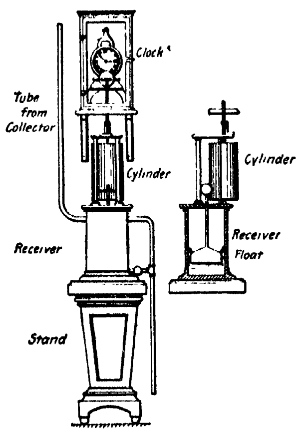


FIG. 83.—FitzGerald gage.

The receiving part of the instrument is of comparatively small size, Fig. 81, and can be set upon the ground.

The principal disadvantages of this gage outside of the very considerable expense, are the delicate mechanisms and electrical contacts to be kept in order. This gage is only made upon special order.

FitzGerald Gage.—This gage was devised in 1878 by Mr. Desmond FitzGerald and was described by him in *Engineering News*, May 31, 1884. The rain is collected in a funnel 14.85 in. in diameter, and conducted through a tube into a receiver containing a float. The diameter of the receiver is such that 1 in. of rain causes the float to rise 2 in. The float carries a pencil bearing directly upon the chart carried by a revolving cylinder. This cylinder is of such a size that a chart 24 in. long is revolved once every day so that the time-scale is 1 in. per hour. It is therefore possible to determine rates of precipitation with fair accuracy.

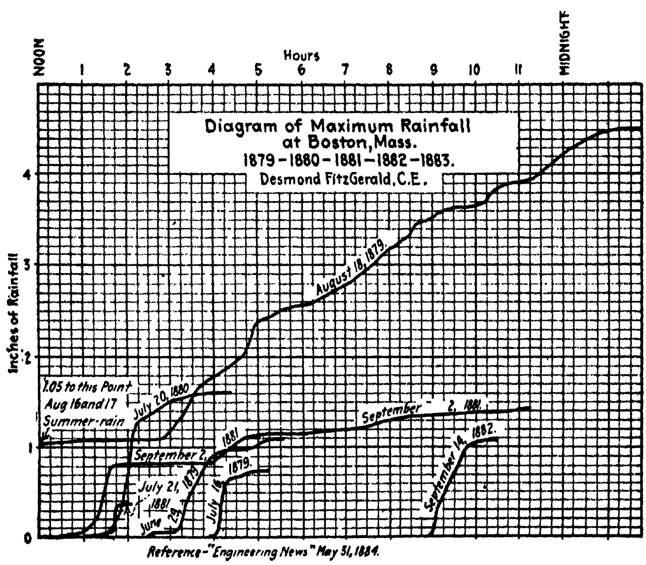
This instrument has never been put upon the market, but it has been used by the Boston and Metropolitan Water Works at Chestnut Hill Reservoir, and the Engineer Department of the District of Columbia.[1]

The principal disadvantage of this type of gage is that its collector must be placed upon the roof of a building, or else a chamber partly underground must be constructed to contain the receiver and cylinder.

[1] The Builders Iron Foundry, of Providence, R. I., is preparing designs for a gage of the FitzGerald type, which it is expected will be sold for about $100.

It has the great advantages that it requires no electrical apparatus and no mechanical motions other than the clockwork.

Hellmann Gage.—This type of recording rain-gage is commonly used in Germany and in other parts of Europe. It is similar to the FitzGerald gage in that the recording pen is carried directly by an arm connected with a float in the reservoir, and makes its record upon a revolving cylinder. It differs in the small size of the reservoir and drum, the former making necessary some appliance for emptying the receiver for comparatively small accumulations of water. In this gage a siphon is provided, by which the receiver is emptied into a can below, the contents of which can afterward be measured. The chart record is seriously defective on



The complete diagram is 24 inches long.

FIG. 84.—Chart from FitzGerald gage.

account of its small time-scale. This gage has the further objection that whatever rain falls while the receiver is being siphoned is not suitably recorded, and serious errors may be introduced from this source.

CLOCK MOVEMENTS

The importance of a good clock movement in an automatic rain gage is not always recognized. It is, however, a point that should receive

careful consideration in the selection of an instrument. Not only should the clock be carefully regulated to keep correct time, but it is of much importance in any work involving two or more automatic gages, that the clocks be accurately synchronized, as otherwise it is impossible to draw any trustworthy conclusions relating to travel of storms, or to the time interval between rainfall and run-off in sewers. It is very desirable that all clocks be provided with dials to facilitate regulating and synchronizing, and in important work of large extent the practicability of electrically controlled clocks should be considered.

SETTING AND EXPOSURE OF GAGES

The correct setting of an automatic rain gage is also of great importance. The exposure should be such that no objects which might interfere with the registration, by causing wind currents or otherwise, are within 50 ft. of the gage, and the collector ring or opening of the gage should not be more than 30 in. above the surface of the ground, which is the standard setting for the regular Weather Bureau rain-gage. This last condition is one which it is often difficult to meet. Elevated gages usually show a considerably less collection than those with standard setting. If it is impossible to locate an automatic gage substantially at ground level, a standard rain-gage of the ordinary type should be maintained in the vicinity of the automatic gage, and the records of the latter should be adjusted to accord with those of the standard gage. The following paragraphs from Circular E, Instrument Division of the U. S. Weather Bureau, entitled "Measurement of Precipitation," are pertinent in this connection:

"**Exposure of Gages.**—The exposure of gages is a very important matter. The wind is the most serious disturbing cause in collecting precipitation. In blowing against the gage the eddies of wind formed at its top and about the mouth carry away rain, and especially snow, so that too little is caught. Snow is often blown out of a deep gage after once lodging therein.

"Rain-gages in slightly different positions, if badly exposed, catch very different amounts of rainfall. Within a few yards of each other two gages may show a difference of 20 per cent. in the rainfall in a heavy rain storm. The stronger the wind the greater the difference is apt to be. In a high location eddies of wind produced by walls of buildings divert rain that would otherwise fall in the gage. A gage near the edge of the roof, on the windward side of a building, shows less rainfall than one in the center of the roof. The vertical ascending current along the side of the wall extends slightly above the level of the roof, and part of the rain is carried away from the gage. In the center of a large, flat roof, at least 60 ft. square, the rainfall collected by a gage does not differ materially from that collected at the level of the ground. A gage on a plane with a tight board fence 3 ft. high around it at a distance of 3 ft. will collect 6 per cent. more rain than if there were no fence. These differences are due entirely to wind currents.

"Since the value of the precipitation records depends so greatly upon proper exposure, particular care should be taken in selecting a place for the location of the gage, and every precaution should be taken to protect it from molestation. If possible, a position should be chosen in some open lot, unobstructed by large trees or buildings. Low bushes and fences, or walls that break the force of the wind in the vicinity of the gage, are beneficial, if at a distance not less than the height of the object. The gages should be exposed upon the roof of a building only when a better exposure is not available; and, when so located, the middle portion of a flat, unobstructed roof enclosed by parapet walls generally gives the best results.

"**The Absolute Measurement of Rainfall.**—It is generally conceded that the true catch of rainfall is obtained by the so-called pit gage; that is, a sunken collector, with its mouth elevated above the ground only far enough to prevent insplashing to any serious extent and set in the middle of a large open level field. Such a gage, however, easily becomes fouled with leaves and litter, and consequently its use is objectionable except as a standard of reference in experimental investigations. A better disposition is secured by forming a shallow pit, a foot or so deep, with the earth thrown up in a circular rim 6 or 8 ft. in diameter. The collector is placed at the center of the depression with its mouth about level with, or a little below the rim of the pit. Such a gage is so effectually sheltered from the wind that it collects the same quantity of rain as falls upon an equal area of the ground near by.

"Nipher demonstrated in 1878 that almost or quite the true catch of rainfall could be collected in ordinary rain-gages by surrounding them with a trumpet-shaped shield of sheet metal terminated in an annular rim of copper wire gauze, 20 gage, mesh 8 wires per inch, to prevent insplashing. This device so far minimized the ill effects of the wind, that one of these shielded gages on a pole 18 ft. above the tower of the university and 118 ft. above the ground, collected the same amount of rainfall as a shielded gage on the ground. Hellmann and others have also found the Nipher screen useful, and have secured equally satisfactory results by the use of a fence or wind brake around the top of the gage, at a distance from it equal to the height of the gage, and at an angular elevation above the gage of about 20 to 30 deg. These devices deflect and check the force of the wind at the mouth of the gage to such an extent that the raindrops can enter the gage in a normal manner, and a true catch be obtained.

"It seems appropriate at this point to say that, while the Weather Bureau is compelled to expose rain-gages upon the roofs of lofty buildings in large cities, the catch of rainfall thus obtained is often quite satisfactory. This is accomplished by taking advantage of the sheltering and protecting influences afforded by large parapet walls, which are generally to be found around flat-topped office buildings. Shields and guards upon the gages themselves in these cases are not so effectual, since the distribution of the rain over the roof is quite irregular. The whole building may be regarded as a huge, lofty rain-gage. If shields and fences could be put around the building itself, a true catch might be secured, but in the absence of these, a gage located in the middle of the roof, especially if it is surrounded by parapet walls 3 or 4 ft. high, collects nearly the true amount of rain. Roof exposures are accepted by the Weather Bureau as an unavoidable necessity at its

stations in the centers of large cities where better exposures are impossible Ground exposures obtain wherever conditions permit, as for example in the smaller cities and at stations of cooperative and special rainfall observers.

"From what has been stated it appears that the pit-gage is probably the ultimate standard for the collection of rainfall and that a nearly true catch may also be obtained by the use of properly shielded gages."

INTENSITY OF PRECIPITATION

It is well known that in a general way the intensity of precipitation varies inversely with the duration of the downpour, or in other words, that very heavy showers do not last as long as rains of lesser intensity. This variation in intensity, however, was not widely recognized as significant until automatic rain gages had been used to a considerable extent. Until recent years no considerable amount of trustworthy information on intensity of precipitation was available, since practically all rainfall records included little more than the total precipitation in each storm, or at most the time of beginning and ending of the storm in addition to the depth of rain. Moreover, not until the establishment of automatic or self-recording rain-gages became somewhat general, and until these had been maintained for a sufficient period to get records of some length, was there sufficient information on which to predicate definite statements as to the relation between the intensity of rainfall and the length of the period during which the rain might fall continuously at any given rate.

Relation between Intensity and Duration of Rain.—One of the earliest attempts to determine the relation between the intensity and duration of precipitation was that of Prof. F. E. Nipher of St. Louis, who, studying the records for that city, for a period of 47 years, and analyzing such information as he could find relating to the heavier storms, reached the conclusion that this relation could be shown by the formula $i = 360/t$ (*The American Engineer*, May 8, 1885), where i is the rainfall in inches per hour and t is the duration of the rainfall in minutes.

In 1889, Emil Kuichling, investigating the rainfall in the vicinity of Rochester, N. Y., similarly studied such records as were available, and reached the conclusion that in Rochester, for rains lasting less than 1 hour the intensity might be expressed by the formula $i = 3.73 - 0.0506\,t$, and for periods longer than 1 hour and less than 5 hours, the intensity would be $i = 0.99 - 0.002t$. To Kuichling's studies is due, in large measure, the present development of the rational method of estimating storm-water run-off.

In 1891 Prof. A. N. Talbot analyzed in detail the rainfall records reported by the United States Weather Bureau and some from other sources. The greater part of them were records of ordinary rain-gages,

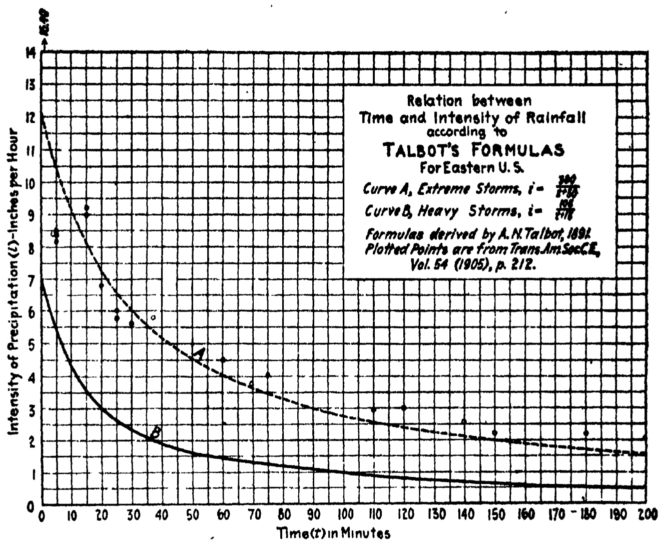


Fig. 85.—Talbot's intensity of rainfall curves.

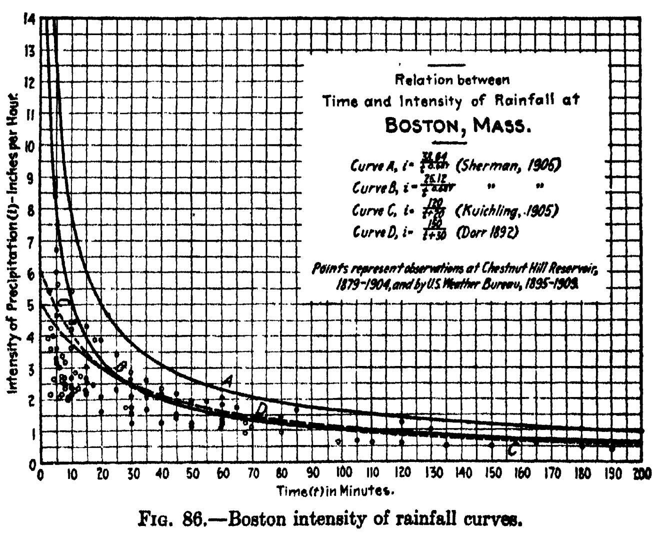


Fig. 86.—Boston intensity of rainfall curves.

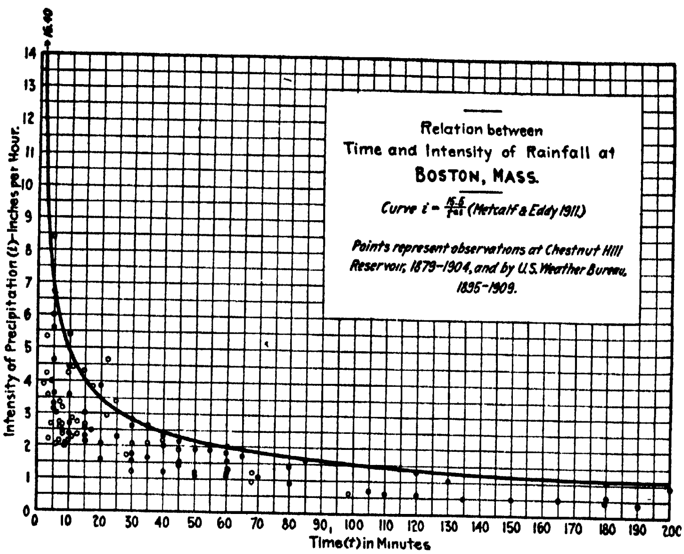


FIG. 87.—Boston intensity of rainfall curve.

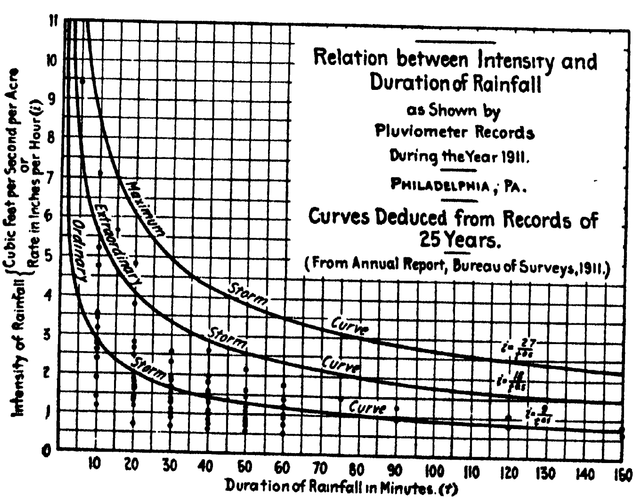


FIG. 88.—Philadelphia intensity of rainfall curves.

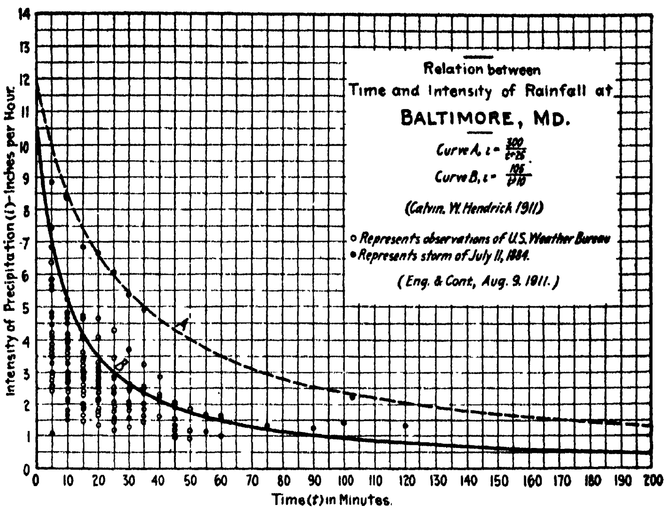


FIG. 89.—Baltimore intensity of rainfall curves.

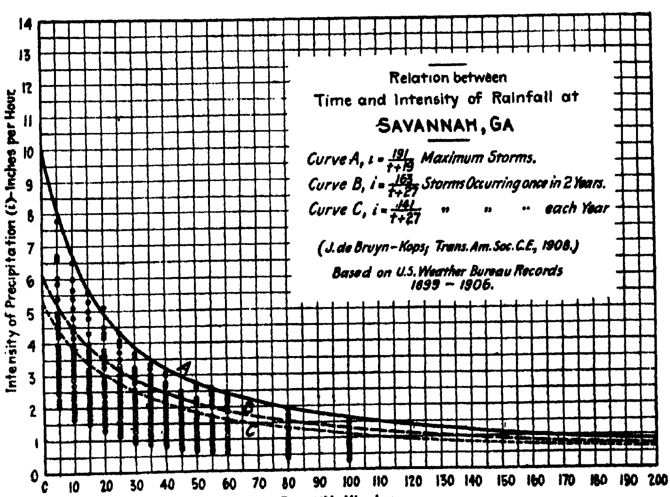


FIG. 90.—Savannah intensity of rainfall curves.

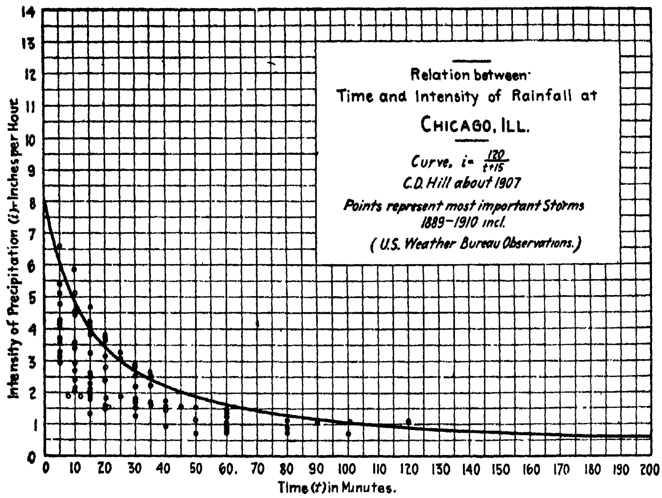


FIG. 91.—Chicago intensity of rainfall curve.

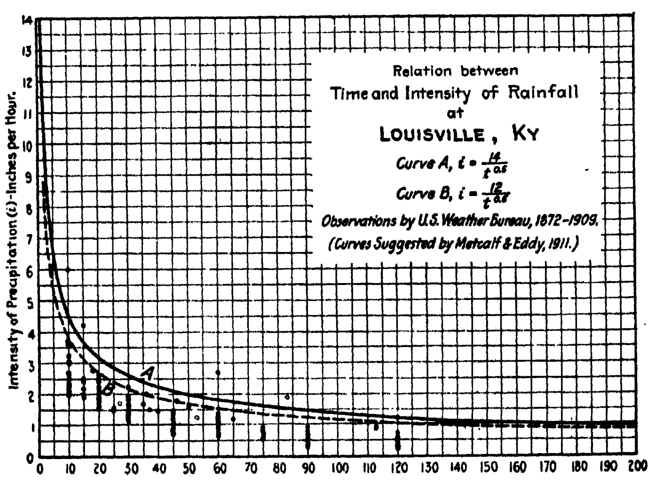


FIG. 92.—Louisville intensity of rainfall curves.

but in a few cases the records were those of self-recording gages maintained in the larger cities. From this study he concluded that for that part of the United States lying east of the Rocky Mountains the formula $i = 360/(t + 30)$ would express the maximum rainfall which was ever likely to occur, and the formula $i = 105/(t + 15)$ would indicate the intensity of as heavy rains as it would ordinarily be necessary to consider in engineering design. Talbot's studies show very few storms indeed giving intensities higher than those shown by the first formula, but a considerable number higher than the second equation.

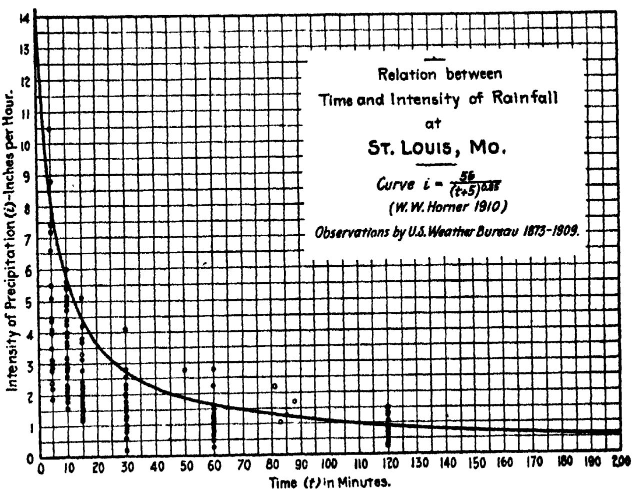


FIG. 93.—St. Louis intensity of rainfall curve.

More recently, with the increasing use of automatic rain-gages, and with the greater use of the rational method of design among sewer engineers, the records of automatic rain-gages in the more important cities have been separately analyzed in detail, and curves have been prepared which have been used as a basis of design in those cities. The curves shown in Figs. 85 to 97, inclusive, have been selected as typical examples of such curves.

St. Louis, Mo.—Fig. 93 shows the rainfall curve derived for St. Louis by W. W. Horner, Assistant Engineer in charge of Sewer Design, together with the observations upon which it is based. The method by which this curve was derived is thus described by Mr. Horner in *Engineering News*, Sept. 29, 1910:

"In Fig. 94 are shown the Weather Bureau Records of excessive rains in St. Louis. The abscissas are the years of occurrence and the ordinates are the rates of rainfall. Each point represents a rain; those points in which small arrows are shown indicate that the value was derived from a rain of slightly greater duration, and that the rate shown is therefore in error by a small amount. From these graphs, values were chosen for the

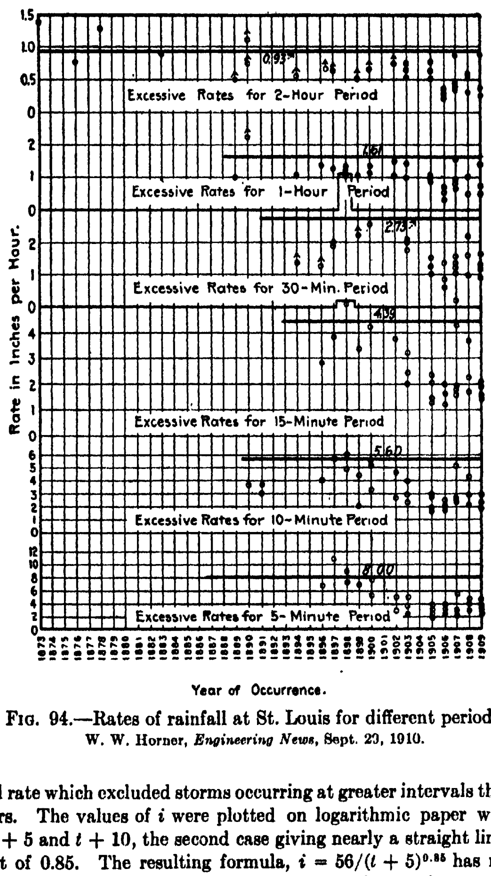


FIG. 94.—Rates of rainfall at St. Louis for different periods.
W. W. Horner, *Engineering News*, Sept. 29, 1910.

rainfall rate which excluded storms occurring at greater intervals than about 15 years. The values of i were plotted on logarithmic paper with those for t, $t + 5$ and $t + 10$, the second case giving nearly a straight line, with a tangent of 0.85. The resulting formula, $i = 56/(t + 5)^{0.85}$ has rather an awkward form, but fits the values chosen so closely that it was retained."

The plotted storms show no warrant for the curve, $i = 360/t$, by Prof. Nipher in 1885.

Spokane, Wash.—R. A. Brackenbury, in *Eng. Record*, Aug. 10, 1912, gives the formula

$$i = \frac{23.92}{t + 2.15} + 0.154$$

which was used in the design of a large storm-water sewer for Spokane.

Comparison of Curves.—A comparison of the several curves shown above is given in tabular form in Table 69, which also includes the general curve $i = 32/t^{0.8}$ suggested by Charles E. Gregory, and the four general formulas

$$i = \frac{8}{t^{0.5}} \qquad i = \frac{12}{t^{0.5}}$$

$$i = \frac{10}{t^{0.5}} \qquad i = \frac{15}{t^{0.5}}$$

Form of Rainfall Curve.—In many cases where mathematical expressions have been obtained for the rainfall curve, they have been written in

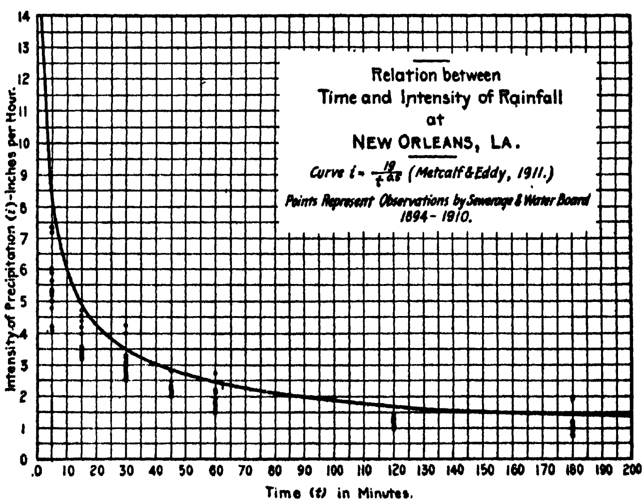


FIG. 95.—New Orleans intensity of rainfall curve.

the form, $i = a/(t + b)$. This formula has the advantage of being easily solvable by simple arithmetical operations; and if the constants in it have been correctly determined, it usually expresses the actual observations with a fair degree of accuracy between the limits of 10 minutes and 2 hours' duration of rain. For either greater or less periods of time, however, the results obtained from this form of curve are generally too

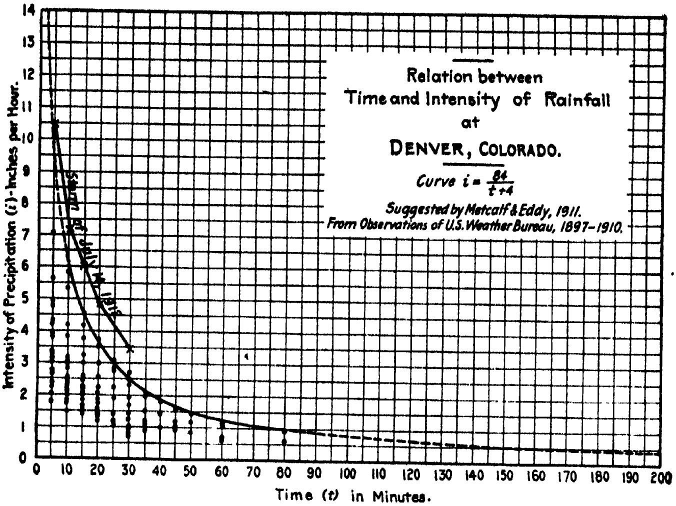


FIG. 96.—Denver intensity of rainfall curves.

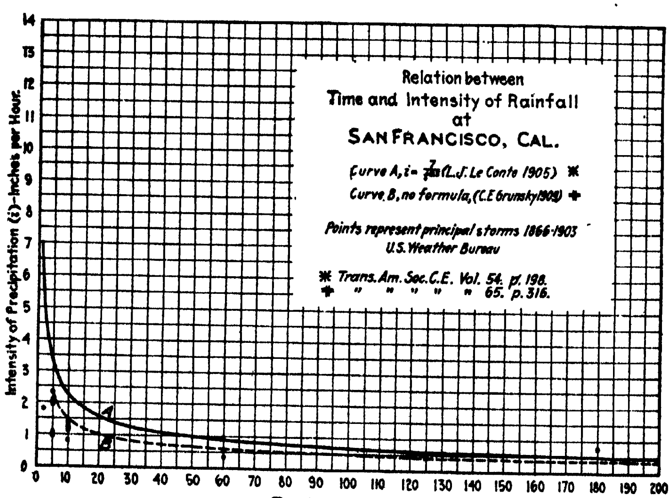


FIG. 97.—San Francisco intensity of rainfall curves.

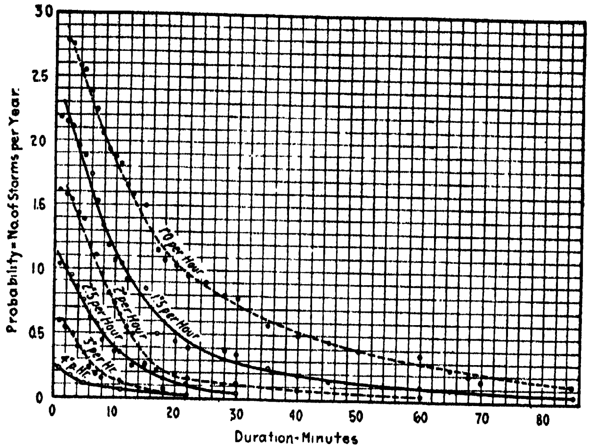


FIG. 98.—Probability of the occurrence at Boston in any year of rainfalls of at least the given intensity for at least the stated time.

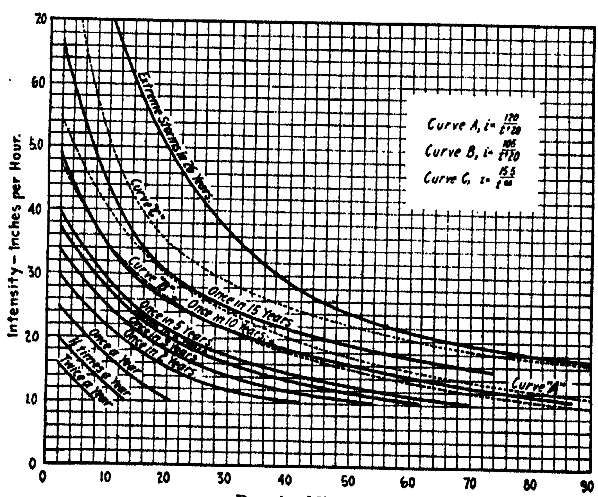


FIG. 99.—Curves of rainfall at Boston of various degrees of probability.

TABLE 69.—COMPARISON OF RAINFALL CURVES

Locality	Formula	Proposed by	Conditions represented by formula	Time t in minutes: 5	10	15	20	30	45	60	90	120	180	240	300	600
				Intensity of precipitation i, inches per hour												
Eastern U. S.	$i=\frac{360}{t+30}$	Talbot	Maximum	10.29	9.00	8.00	7.20	6.00	4.80	4.00	3.00	2.40	1.72	1.33	1.09	0.57
Eastern U. S.	$i=\frac{105}{t+15}$	Talbot	Ordinary	5.25	4.20	3.50	3.00	2.33	1.75	1.40	1.00	0.78	0.54	0.41	0.33	0.17
Boston, Mass.	$i=\frac{150}{t+30}$	Dorr	Basis of design	4.28	3.75	3.33	3.00	2.50	2.00	1.67	1.25	1.00	0.71	0.56	0.45	0.24
Boston, Mass.	$i=\frac{106}{t+13}$	Kuichling	Basis of design	5.89	4.61	3.79	3.21	2.46	1.83	1.44	1.03	0.80	0.55	0.42	0.34	0.17
Boston, Mass.	$i=\frac{120}{t+20}$	Kuichling	Basis of design	4.80	4.00	3.43	3.00	2.40	1.85	1.50	1.09	0.86	0.60	0.46	0.38	0.19
Boston, Mass.	$i=38.64\div t^{0.687}$	Sherman	Maximum	12.80	7.95	6.01	4.94	3.74	2.82	2.31	1.75	1.44	1.09	0.90	0.75	0.48
Boston, Mass.	$i=25.12\div t^{0.687}$	Sherman	Basis of design (max.)	8.32	5.17	3.91	3.21	2.43	1.84	1.51	1.14	0.93	0.71	0.60	0.50	0.31
Boston, Mass.	$i=15.5\div t^{0.5}$	Metcalf & Eddy	Basis of design (max.)	6.93	4.90	4.09	3.47	2.83	2.31	2.00	1.63	1.41	1.16	1.00	0.90	0.63
Philadelphia, Pa.	$i=27\div t^{0.5}$	Bureau of Surveys	Maximum	12.07	8.54	6.99	6.04	4.94	4.02	3.49	2.85	2.46	2.01	1.74	1.56	1.10
Philadelphia, Pa.	$i=18\div t^{0.5}$	Bureau of Surveys	High	8.05	5.69	4.65	4.03	3.29	2.68	2.32	1.90	1.64	1.34	1.16	1.04	0.73
Philadelphia, Pa.	$i=9\div t^{0.5}$	Bureau of Surveys	Ordinary	4.02	2.84	2.32	2.01	1.64	1.34	1.16	0.95	0.82	0.67	0.58	0.52	0.37
Baltimore, Md.	$i=\frac{300}{t+25}$	Hendrick	Maximum	10.00	8.58	7.50	6.67	5.47	4.29	3.54	2.61	2.07	1.46	1.12	0.90	0.48
Baltimore, Md.	$i=\frac{105}{t+10}$	Hendrick	Basis of design	7.00	5.25	4.20	3.50	2.63	1.91	1.50	1.05	0.81	0.58	0.42	0.34	0.17
Savannah, Ga.	$i=\frac{191}{t+19}$	de Bruyn-Kops	Maximum	7.95	6.58	5.61	4.89	3.89	2.98	2.42	1.75	1.38	0.96	0.74	0.60	0.31
Savannah, Ga.	$i=\frac{163}{t+27}$	de Bruyn-Kops	Once in 2 years	5.10	4.41	3.88	3.47	2.86	2.27	1.88	1.40	1.11	0.79	0.61	0.50	0.26
Savannah, Ga.	$i=\frac{141}{t+27}$	de Bruyn-Kops	Once a year	4.41	3.82	3.36	3.00	2.47	1.96	1.63	1.21	0.96	0.68	0.53	0.43	0.23
Chicago, Ill.	$i=\frac{120}{t+15}$	Hill		6.00	4.80	4.00	3.42	2.67	2.00	1.60	1.14	0.89	0.62	0.51	0.38	0.20
Louisville, Ky.	$i=14\div t^{0.5}$	Metcalf & Eddy		6.26	4.42	3.61	3.13	2.56	2.08	1.81	1.48	1.28	1.04	0.91	0.81	0.57
New Orleans, La.	$i=19\div t^{0.5}$	Metcalf & Eddy		8.50	6.00	4.91	4.25	3.47	2.83	2.45	2.00	1.73	1.42	1.23	1.09	0.78
St. Louis, Mo.	$i=56\div (t+5)^{0.85}$	Horner		8.00	5.60	4.39	3.63	2.73	2.01	1.61	1.17	0.93	0.66	0.47	0.39	0.24
Denver, Colo.	$i=84\div (t+4)$	Metcalf & Eddy		9.33	6.00	4.42	3.50	2.47	1.71	1.31	0.90	0.68	0.46	0.34	0.28	0.14
San Francisco, Cal.	$i=3.68\div\left[\frac{2t}{t+60}+t^{0.4}\right]$	Grunsky		1.79	1.31	1.10	0.97	0.81	0.68	0.60	0.51	0.45	0.38	0.34	0.32	0.25
San Francisco, Cal.	$i=5\div t^{0.5}$	Metcalf & Eddy		2.24	1.58	1.29	1.12	0.91	0.75	0.65	0.53	0.46	0.37	0.32	0.29	0.20
Spokane, Wash.	$i=23.92\div (t+2.15)+0.154$	Brackenbury		3.49	2.12	1.55	1.24	0.90	0.66	0.54	0.41	0.35	0.29	0.25	0.23	0.19
Berlin, Germany	$i=\frac{25.6}{t}+0.61$	Knauff		5.73	3.17	2.31	1.89	1.46	1.18	1.04	0.89	0.82	0.75	0.72	0.70	0.65
General formulas	$i=32\div t^{0.8}$	Gregory		8.86	5.07	3.67	2.92	2.10	1.52	1.21	0.87	0.69	0.50	0.40	0.33	0.19
	$i=8\div t^{0.5}$	Metcalf & Eddy		3.58	2.53	2.06	1.79	1.46	1.19	1.03	0.84	0.73	0.60	0.52	0.46	0.33
	$i=10\div t^{0.5}$	Metcalf & Eddy		4.47	3.16	2.58	2.23	1.82	1.49	1.29	1.05	0.91	0.75	0.65	0.58	0.41
	$i=12\div t^{0.5}$	Metcalf & Eddy		5.36	3.80	3.10	2.68	2.19	1.79	1.55	1.26	1.09	0.90	0.78	0.69	0.49
	$i=15\div t^{0.5}$	Metcalf & Eddy		6.71	4.75	3.88	3.36	2.74	2.24	1.94	1.58	1.37	1.12	0.97	0.87	0.61

low. Practically, this is usually of little moment, since it is rarely necessary to consider a shorter period of time than 10 minutes or a greater one than 2 hours in the design of sewers. It is, however, desirable, if the curve is to be expressed in mathematical form, to have this form as nearly correct as possible, and the exponential form, $i = a/t^b$ has been found to fit the recorded observations with a good degree of accuracy in many cases. The exponent b is usually found to be between 0.5 and 0.7. When it is just 0.5, this equation can be solved as easily as the former one, since the results can be obtained directly with a single setting of the slide rule for each value of b.[1]

The form $i = a/\sqrt{(t + b)}$ is also a convenient one for use, as it can be solved nearly as easily as the equation just mentioned, and at least in some cases it fits the observed points more satisfactorily. It has the further advantage that the value of i as it approaches zero is a finite quantity, its magnitude depending on the values chosen for a and b.

The best practice, however, is probably that of using directly a curve of rainfall plotted from the actual records, without attempting to express it in mathematical language. The intensity to be expected for any given duration would then be taken directly from the curve, instead of being obtained by solving some equation. There is no apparent reason why the relation between the intensity and duration should follow any mathematical law.

When rainfall records from which to construct a curve are not to be had, it is believed that for the portion of the United States lying east of the Mississippi River, the formula $i = a/t^{0.5}$ may be used as a reasonable basis of design, with values of a ranging from 15 to 10, depending upon local conditions and upon the extent to which occasional surcharging or flooding of the works under consideration may be allowed. For conditions similar to those existing in New England and New York, it is believed that in general the formula $i = 12/t^{0.5}$ will indicate intensities that are not likely to be exceeded oftener than about once in 10 years.

FREQUENCY OF HEAVY STORMS

It is of much importance to have at least an approximate knowledge of how often storms of high intensities are likely to occur. Where the rainfall record covers a considerable period of time it is possible to compile such information with a good degree of accuracy. The method of working up the records and expressing the results in graphical form is illustrated in the following example.

A record of the storms of high intensity at Boston, for the 26 years

[1] Set the runner at the value of a on the lower scale of the rule; move the slide until the value of t on the upper scale of the slide is opposite the runner; find i on the lower scale opposite the end of the slide.

TABLE 70.—PROBABILITY OF THE OCCURRENCE IN ANY YEAR OF STORMS OF AN INTENSITY OF 1 INCH PER HOUR OR GREATER, FOR VARIOUS PERIODS OF TIME, AT BOSTON, MASS.

(Based upon the records of the recording rain gage at Chestnut Hill Reservoir, Boston, for 26 years, 1879–1904 inclusive.)

Duration, minutes	Number of storms of 1 in. intensity or greater	Total number of storms of the stated or greater duration of 1 in. or greater intensity	Corresponding number of storms per year, = probability
2	1	72	2.78
3	4	71	2.74
4	1	67	2.58
5	4	66	2.54
6	4	62	2.39
7	5	58	2.24
8	3	53	2.04
9	2	50	1.92
10	1	48	1.85
11	4	47	1.81
12	2	43	1.65
13	2	41	1.58
15	9	39	1.50
17	2	30	1.15
18	1	28	1.08
20	2	27	1.04
22	1	25	0.96
25	3	24	0.92
28	1	21	0.81
30	5	20	0.77
35	2	15	0.58
40	1	13	0.50
45	2	12	0.46
50	1	10	0.39
60	3	9	0.35
65	1	6	0.23
68	1	5	0.19
70	1	4	0.15
85	1	3	0.12
130	1	2	0.08
180	1	1	0.04
	72		

1879–1904 inclusive, is contained in Trans. Am. Soc. C. E., vol. liv, pp. 174–176. From this record Table 70 has been prepared, giving the number of storms of 1 in. or greater intensity, the number of such storms of designated durations and the corresponding number of storms of each duration per year. By plotting the points obtained from each such tabulation, a series of curves is obtained, shown by the diagram, Fig. 98.

For practical use, it is generally more helpful to know the curve of intensity of precipitation in storms of various degrees of frequency. From Fig. 98 it is seen that for a storm of a frequency of unity, or such as is likely to occur once each year, an intensity of 2.5 corresponds to a duration of 2 minutes; an intensity of 2 to a duration of 7 minutes; and so on. A series of curves of intensity corresponding to different degrees of frequency or probability can readily be constructed in this way. Such curves for Boston are shown in Fig. 99, together with three curves represented by some of the formulas which have been proposed.

TABLE 71.—PHENOMENAL RAINFALLS IN NEW YORK CITY, 1913

Date	July 10	July 28	Sept. 5	Oct. 1	
Place	100 Broadway	Central Park	Central Park	Richmond	$i = \frac{15}{\sqrt{t}}$
t-minutes	Intensity i-inches per hour				
1	......	......	......	8.40	15.00
2	......	......	......	8.10	10.60
4	......	......	......	6.45	7.50
5	9.88	6.12	7.20	......	6.72
7	......	......	......	6.24	5.68
10	7.56	5.76	6.90	5.64	4.75
15	6.52	4.80	6.36	5.16	3.88
19	......	......	......	5.05	3.45
30	4.18	2.96	5.24	......	2.74
37	......	......	......	4.84	2.47
49	......	......	......	4.75	2.15
59	......	......	......	4.44	1.95
60	2.30	2.73	3.31	......	1.94
85	......	......	......	3.80	1.63
106	......	......	......	3.37	1.46
120	1.28	1.56	1.85	......	1.37
123	......	......	......	3.06	1.35

From this diagram it is apparent that the curve of extreme storms is governed by some very abnormal cases. The curve C, represented by the equation $i = 15.5/t^{0.5}$, may doubtless be taken to represent storms occurring not oftener than once in twenty years, and should be a safe basis of design for even most important structures. The curves A and B, represented by the equations $i = 120/(t+20)$ (proposed by Kuichling in 1905) and $i = 105/(t+20)$, respectively, correspond reasonably well

with the intensity curves of storms to be expected once in 15 and once in 10 years, respectively, as do also the curves $i = 12/t^{0.5}$ and $i = 10/t^{0.5}$, respectively, and should be appropriate bases of design for the less important structures such, perhaps, as the branches of a drainage system.

Similar curves of rainfall intensity and frequency for the City of New York have appeared as this book is passing through the press in the 1913 progress report of the Committee on Rainfall and Run-off of the Society of Municipal Engineers of the City of New York. Intensity curves are given for several localities. The frequency curves are based upon the 45-year record of the recording rain-gage in Central Park.

Phenomenal Rainstorms.—Storms of extreme intensity, commonly called "cloud-bursts," are occasionally experienced in the Eastern United States. They are usually of so rare occurrence as to be classed as "Acts of God," for which it would not be reasonable to provide in designing storm sewers.

During 1913 New York City experienced four storms, in all of which the intensity of precipitation, practically throughout the storm, was greater than that given by the equation $i = 15/t^{0.5}$. The significant facts relative to these storms and intensities obtained by this formula are contained in Table 71.

These were all storms of remarkable intensity. The maximum rates attained may be expressed approximately by the formula $i = 35/\sqrt{(t+7)}$. These rates approximate those given by Talbot's "maximum" curve for the shorter periods, but exceed them materially for longer periods of time.

CHAPTER VII

FORMULAS FOR ESTIMATING STORM-WATER FLOW

In the earlier plans for drains and channels to carry away the water of storms, engineers based their designs largely upon their observations of the volumes of water seen coming from known areas in times of storm and upon the sizes of natural gutters or water-courses with which they were more or less familiar. Later the tributary areas, which could be accurately measured, were introduced as constants, and the estimates of run-off were based upon a given depth of precipitation over the whole district; but with further study it developed that there is a gradual reduction in the immediate run-off per acre with an increase in the extent of the area, and accordingly formulas were devised by which this fact was taken into account more or less empirically. Still more recently it has been recognized that differences in the rainfall, and especially in the intensity of the precipitation, had a direct influence upon the resulting storm-water flow, and other factors have been introduced into the formulas to take account of this and of the slope and dimensions of the drainage area. The result has been the gradual development of a number of empirical formulas or diagrams, by which the greatest quantity of storm water to be discharged from any given drainage area could be estimated.

EMPIRICAL FORMULAS

The best known of these empirical formulas, reduced to a uniform notation, and with the introduction of a term expressing rate of rainfall (which was not originally used in all of them), are as follows:

Hawksley (London, 1857): $Q = ACi\sqrt[4]{(S/Ai)}$, in which $C = 0.7$ and $i = 1.0$, so that $Q = 3.946A\sqrt[4]{s/A}$ (since $s = S/1000$).

Bürkli-Ziegler (Zurich, 1880): $Q = ACi\sqrt[4]{(S/A)}$, in which $C = 0.7$ to 0.9, and $i = 1$ to 3.

Adams (Brooklyn, 1880): $Q = ACi\sqrt[12]{(S/A^2i^2)}$, in which $C = 1.837$ and $i = 1$.

McMath (St. Louis, 1887): $Q = ACi\sqrt[5]{(S/A)}$, in which $C = 0.75$ and $i = 2.75$.

Hering (New York, 1889) $Q = CiA^{0.85}S^{0.27}$, or

$$Q = Aci\sqrt[6]{(S^{1.62}/A)} = CiA^{0.833}S^{0.27}$$

in which Ci varies from 1.02 to 1.64. These two formulas give considerably different results.

Parmley (Cleveland, 1898) $Q = ACi\sqrt[4]{S^{1.5}/A}$, in which C is between 0 and 1, and $i = 4$.[1]

Gregory (New York, 1907) $Q = ACi\,S^{0.186}/A^{0.14}$, in which $Ci = 2.8$ for impervious surfaces.

Where Q = the maximum discharge of sewer in cubic feet per second,
i = maximum rate of rainfall in inches per hour (which is almost the same as the quantity of precipitation in cubic feet per second per acre),[2]

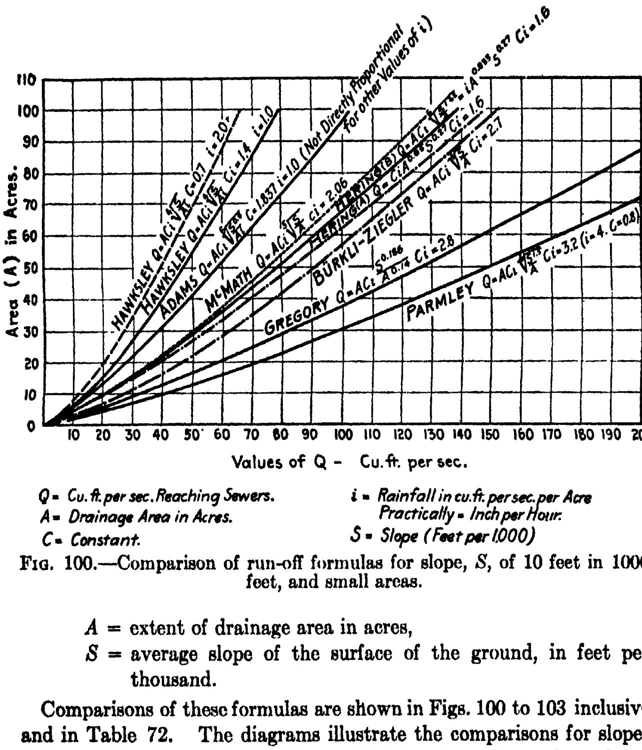


FIG. 100.—Comparison of run-off formulas for slope, S, of 10 feet in 1000 feet, and small areas.

A = extent of drainage area in acres,
S = average slope of the surface of the ground, in feet per thousand.

Comparisons of these formulas are shown in Figs. 100 to 103 inclusive and in Table 72. The diagrams illustrate the comparisons for slopes of 4, 10 and 50 ft. per 1000 ft., and the tabulation covers these slopes and also 250 ft. per 1000 ft. It will be seen that a very wide range of results may be obtained, depending upon the formula chosen.

[1] Parmley takes i as representing the intensity of rainfall for a period of 8 or 10 minutes, and for the Walworth Sewer (Cleveland) used $i = 4$ in order to provide for the most violent storms, and also for the further damage caused by the prevailing direction of the storms.

[2] One inch per hour = 1.008 cu. ft. per second per acre.

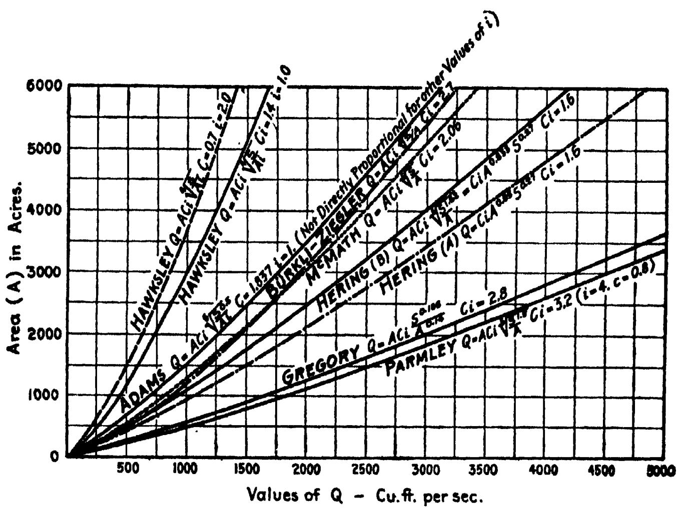
TABLE 72.—COMPARISON OF RESULTS OBTAINED BY USE OF SEVERAL EMPIRICAL FORMULAS FOR RUN-OFF

Slope of surface per 1000	Flat slopes, $S = 4$				Flat slopes, $S = 10$				Medium slopes, $S = 50$				Very steep slopes, $S = 250$			
Area drained (acres)	10	100	1,000	5,000	10	100	1,000	5,000	10	100	1,000	5,000	10	100	1,000	5,000
Formulas	Run-off, cubic feet per second															
Hawksley ($C = 0.7, i = 2$)	9.4	53	296	990	12	66	372	1,247	18	99	557	1,862	26	148	832	2,784
Bürkli-Ziegler ($Ci = 2.7$)	21.5	121	679	2,271	27	152	854	2,855	40	227	1,278	4,270	60	340	1,909	6,384
Adams ($C = 1.837, i = 1$)	14.1	96	652	2,493	15	103	703	2,691	17	118	805	3,078	20	135	920	3,519
McMath ($C = 0.75, i = 2.75$)	17.2	108	683	2,474	21	130	820	2,972	28	179	1,131	4,100	39	247	1,561	5,659
Hering (A) ($Ci = 1.6$)	16.5	117	825	3,240	21	149	1,057	4,150	33	231	1,632	6,411	50	356	2,521	9,902
Hering (B)	15.8	108	734	2,790	20	138	942	3,602	31	213	1,451	5,548	48	329	2,242	8,565
Gregory ($Ci = 2.8$)	26.3	190	1,378	5,499	31	226	1,634	6,520	42	304	2,204	8,796	57	410	2,963	11,867
Parmley ($C = 0.8, i = 4$)	30.8	210	1,431	5,472	39	264	1,800	6,880	58	395	2,692	10,300	87	591	4,024	15,390

Slope of surface, per 1000	Flat slopes, $S = 4$				Flat slopes, $S = 10$				Medium slopes, $S = 50$				Very steep slopes, $S = 250$			
Diameter of sewer	12″	24″	4′	8′	12″	24″	4′	8′	12″	24″	4′	8′	12″	24″	4′	8′
Slope of sewer	.005	.002	.001	.0005	.005	.002	.001	.0005	.005	.002	.001	.0005	.005	.002	.001	.0005
Formulas	Area drained, acres															
Hawksley ($C = 0.7, i = 2$)	1.30	8.9	67	515	0.94	6.5	49.3	380	0.59	3.8	28.9	222	0.32	2.3	16.9	130
Bürkli-Ziegler ($Ci = 2.7$)	0.42	3.0	22	165	0.31	2.0	16.3	122	0.18	1.3	9.6	71	0.11	0.7	5.6	42
Adams ($C = 1.837, i = 1$)	0.96	5.5	34	208	0.88	5.1	31.1	188	0.75	4.3	26.4	161	0.64	3.7	22.5	137
McMath ($C = 0.75, i = 2.75$)	0.68	4.2	28	184	0.41	3.4	22.2	146	0.36	2.2	15.0	98	0.24	1.5	9.9	65
Hering (A) ($Ci = 1.6$)	0.84	4.7	28	162	0.63	3.5	20.5	121	0.38	2.1	12.3	73	0.23	1.3	7.4	57
Hering (B)	0.83	4.8	29	181	0.62	3.6	21.8	133	0.37	2.1	12.9	80	0.22	1.3	7.9	47
Gregory ($Ci = 2.8$)	0.50	2.7	16	91	0.41	2.2	13.0	75	0.29	1.6	9.2	53	0.21	1.1	6.5	37
Parmley ($C = 0.8, i = 4$)	0.38	2.2	13	81	0.28	1.6	10.1	61	0.18	1.0	6.2	38	0.11	0.6	3.8	23

The first four of these run-off formulas were analyzed and compared by Emil Kuichling in an address before the College of Civil Engineering of Cornell University. (Trans. Assoc. Civ. Engs. Cornell Univ., 1893.) This analysis is reproduced herewith, slightly condensed.

Hawksley's Formula.—This appears to have been established at some time between 1853 and 1856, to express analytically the relation between the diameter and slope of a circular outlet sewer and the magnitude of its drainage area, which is embodied in a table (Table 1) prepared in 1852 by John Roe, Surveyor of the Holborn and Finsbury sewers (London), after



Q = Cu. ft. per sec. Reaching Sewers. *i = Rainfall in cu. ft. per sec. per Acre*
A = Drainage Area in Acres. *Practically = Inch per Hour.*
C = Constant. *S = Slope (Feet per 1000)*

FIG. 101.—Comparison of run-off formulas for slope, S, of 10 feet in 1,000 feet and large areas.

numerous observations of their storm discharge. As rains yielding more than 1 in. in depth per hour are of comparatively rare occurrence in London, an intensity of 1 in. per hour was then probably regarded as a maximum for which provision should be made in municipal sewerage work, and the diameters, grades and areas given by Roe were considered as applicable to such intensity. In its original form, Hawksley's formula is (see Report of Commission of Metropolitan Drainage, London, 1857):

$$\log d = \frac{3 \log A + \log N + 6.8}{10}$$

where d = diameter in inches of a circular sewer adapted to carry off the storm water due to a rainfall of 1 in. per hour;

A = magnitude of the drainage area in acres;

N = length in feet in which the sewer falls 1 ft. If we replace N by its equivalent $(1/s)$, where s denotes the sine of the slope of the sewer, and then divest the above expression of its logarithmic form, there follows:

$$d^{10} = 6{,}309{,}574A^3/s$$

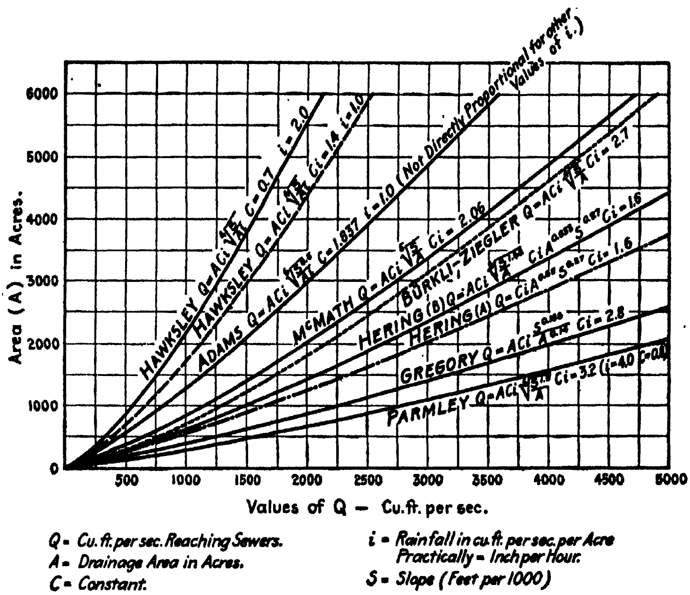


FIG. 102.—Comparison of run-off formulas for slope, S, of 50 feet in 1,000 feet.

and if the diameter is expressed in feet D, instead of in inches d, we will have $d = 12D$, and

$$D^{10} = 0.0001019\frac{A^3}{s} = \frac{A^3}{9813.23s}$$

It must be remembered that A here represents essentially a certain volume of water discharged in a certain period of time by the sewer, and that such volume in cubic feet per second is equal to the number of acres in the drainage area when the entire precipitation, at the rate or intensity of 1 in. per hour, runs off from the surface and reaches the sewer as fast as it falls; also that if the formula contemplates the discharge of only some fraction of this

precipitation, such fraction has presumably been introduced into the constant coefficient. Accordingly, if the intensity i of the rainfall is to be introduced into the formula, the factor A should be replaced therein by the product Ai, which represents the rainfall upon the area in cubic feet per second, thus giving:

$$D^{10} = 0.0001019\frac{A^3i^3}{s}$$

But from the fundamental formula for the flow of water in circular conduits

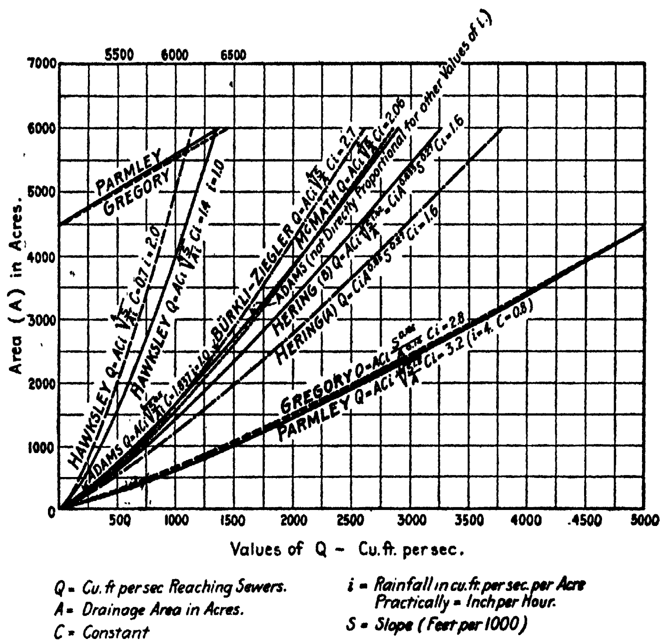


FIG. 103.—Comparison of runoff formulas for slope, S, of 4 feet in 1,000 feet.

running full (the Chezy formula, with $c = 100$) we have the velocity in feet per second, $v = 100\sqrt{Ds/4} = 50\sqrt{Ds}$, and the discharge in cubic feet per second; $Q = \pi D^2v/4$, whence $Q = 39.27\sqrt{D^5s}$, and

$$D^5 = \left(\frac{Q}{39.27}\right)^2\frac{1}{s} \text{ or } D^{10} = \left(\frac{Q}{39.27}\right)^4\frac{1}{s^2}$$

The foregoing two values of D must, however, be equal to each other, whence

$$\left(\frac{Q}{39.27}\right)^4 \frac{1}{s^2} = 0.0001019 \frac{A^3 i^3}{s},$$

and

$$Q = 3.946 A i \sqrt[4]{\frac{s}{Ai}}$$

which is the Hawksley formula.

Bürkli-Ziegler's Formula.—In his paper on "The Greatest Discharge of Municipal Sewers" (Grösste Abflussmengen in Städtischen Abzugskanäle," Zurich, 1880), Bürkli-Ziegler gives the following formula, which is based on Hawksley's expression:

$$q = cr\sqrt[4]{S/A}$$

where q = volume of storm water (liters) reaching the sewer per second from each hectare of the surface drained;
c = empirical coefficient varying with the character of the surface;
r = average rainfall in liters per hectare and per second, during the period of heaviest fall;
S = general grade or fall of the area per thousand;
A = magnitude of drainage area in hectares.

From the data available, the computed values of c ranged from 0.25 for suburban districts, to 0.60 for thickly populated urban districts, with an average value of $c = 0.50$; and for r it is recommended to take values ranging from 125 to 200 liters per hectare per second. Since 1 liter per hectare per second is equivalent to 0.0143 cu. ft. per acre per second, it will be seen that these values correspond to 1.79 and 2.86 cu. ft. per acre per second, or to rain intensities of from 1.79 to 2.86 in. per hour. If we take the volumes q and r in cubic feet per acre per second, the area A in acres, and introduce the sine of the general slope s in place of the grade per thousand S, we will have; $S = 1000\,s$, and

$$q = cr\sqrt[4]{s/A}$$

where the value of c will range from 1.76 to 4.22, with an average value of 3.52; and if we further substitute the total discharge Q for the discharge per acre q, and replace r by its equivalent intensity of rainfall i in inches per hour, there follows; $Q = Aq$ and

$$Q = cAi\sqrt[4]{(s/A)} = ACi\sqrt[4]{(S/A)}$$

as given above.

Adams' Formula.—The formula of Col. J. W. Adams is developed in his book on "Sewers and Drains for Populous Districts" (New York, 1880), from the fundamental expression for the diameter of a circular conduit running full, viz.:

$$D^5 = \left(\frac{Q}{39.27}\right)^2 \frac{1}{s} = \frac{Q^2}{1542s}$$

by arbitrarily changing the exponent of D from 5 to 6, and then substituting $A/2$ for Q on the assumption that one-half of a precipitation $i = 1$ in. per hour will reach the sewer during this period of time, thus giving:

$$D^6 = \frac{A^2}{6168s}, \text{ or } D = \sqrt[6]{\left(\frac{A^2}{6168s}\right)}$$

This change in the exponent of D was made for the purpose of getting a larger value for the run-off Q.

For any other value of i than $i = 1$, we would have to substitute $Ai/2$ for Q thus obtaining;

$$D = \sqrt[6]{\left(\frac{A^2i^2}{6168s}\right)}$$

But for the flow in the conduit we also have

$$D = \sqrt[5]{\left(\frac{Q^2}{1542s}\right)}$$

And as the two values of D must be equal, there follows

$$\sqrt[6]{\frac{A^2i^2}{6168s}} = \sqrt[5]{\frac{Q^2}{1542s}}$$

whence

$$Q = 1.035\,Ai\sqrt[12]{\left(\frac{s}{A^2i^2}\right)}$$

as above.

McMath's Formula.—The formula of R. E. McMath of St. Louis, Mo., was published in 1887 by its author in Trans. Am. Soc. C. E., Vol. XVI, p. 183. Its original form is the same as given above, except that for s the fall S in feet per thousand was used. It seems to have been derived from a number of observations of depth of flow in a variety of sewers of known size and grade draining areas of known magnitude, but apparently without exact knowledge of the maximum intensity of the rainfall which produced the computed discharge, or of the proportion of water reaching the sewers at the period of maximum flow. The discharges were plotted on a diagram as ordinates to the corresponding values of the drainage area as abscissas, whereupon the enveloping curve of the points thus obtained was drawn and its equation sought. This equation appears to have the form of

$$Q = b\sqrt[5]{A^4},$$

and by introducing the average surface grade, the rate of precipitation in cubic feet per acre per second (or the rainfall intensity in inches per hour), and the proportion e of water flowing off from the surface, as factors making up the coefficient b, we may write:

$$Q = ei\sqrt[5]{(SA^4)} = eAi\sqrt[5]{(S/A)}$$

For the city of St. Louis, McMath adopted for these factors, the value $e = 0.75$, $i = 2.75$, and $S = 15$. If, however, it is desired to introduce the

sine of the slope (s) into the expression instead of the grade or fall in feet per thousand, we must substitute for S its value, $S = 1000s$; and by placing $C = e\sqrt[5]{1000}$ there follows:

$$Q = CAi\sqrt[5]{(s/A)}$$

For $e = 0.75$, the value of C will become 2.986, which is presumably applicable to first class urban districts; but for suburban districts the proportion e of the rainfall which reaches the sewers is manifestly smaller, and may be taken at about $e = 0.31$, thus giving $C = 1.234$.

It may be of interest to ascertain which one of the various indexes of the radical $\sqrt[x]{(1/A)}$ in the above formulas is probably the most correct from a theoretical point of view. For this purpose, let us consider the motion of a material point in sliding down an inclined plane or line whose length is l and angle of inclination a. Neglecting frictional resistances, the time t required for such a point to traverse the length l by the action of gravity alone will be, $t = \sqrt{(2l/g \sin a)}$, where g denotes the acceleration of gravity. If we now regard the length l as the path traveled by a particle of water in its passage from the margin of the drainage area through the gutters or smaller sewers to the point of observation in the outlet sewer, then for different values of l on the same slope a, the time t will vary with $\sqrt{l}$. For similar areas A, however, the length l will vary with $\sqrt{A}$; hence the time t will vary with $\sqrt[4]{A}$; and if it be further assumed that this time t is proportional to the retardation of the discharge, or to the ratio of the sewer discharge Q to the precipitation in cubic feet per second R upon the area A, there follows:

$$Q/R = m/t = n\sqrt[4]{A}$$

where m and n are empirical coefficients.

Theoretically, therefore, the fourth root of the factor $1/A$ is the most proper one to use in formulas of the class above described; and where deviations from this rule have been made, in order to accommodate the value of Q to certain observations or measurements, it is fair to conclude that the formula cannot be of general applicability.

Hering Formula (New York Diagrams.)—Diagrams of run-off to be expected in New York City were prepared in 1889 by Rudolph Hering in connection with an unpublished report. In the report of the Baltimore Sewerage Commission, 1897, he and Samuel M. Gray give as a formula deduced from these diagrams: $Q = CiA^{0.85}S^{0.27}$, and this formula is also quoted in Ogden's "Sewer Design."

From the same diagrams, Charles E. Gregory in 1907 (Trans. Am. Soc. C. E., Vol. 58, p. 458) with Hering's report of 1889 before him, deduced the formula as

$$Q = CiA^{0.833}S^{0.27}$$

Both forms of this formula are somewhat widely known. As is shown by Figs. 100 to 103 inclusive, the differences in the results obtained by the employment of the two forms are considerable, amounting to approximately 15 per cent.

Parmley's Formula.—This formula was developed by W. C. Parmley in his studies of conditions in the City of Cleveland, preparatory to designing the large intercepting sewer known as the Walworth Sewer. These studies are described in Jour. Assoc. Eng. Soc., vol. 20, p. 204. where the formula is written in the form

$$Q = Ci\sqrt[4]{S}\,A^{5/6}$$

Parmley concluded that i, the rate of rainfall corresponding to the time required for concentration at the sewer inlets, should be taken at 4 in. per hour.

Gregory's Formula.—In one sense, it is hardly fair to include the Gregory formula among those of empirical derivation, since it is based upon the rational formula $Q = CiA$. It is, however, interesting to compare the results obtained by his method and assumption, with those based upon the use of the empirical formulas.

As explained at length in Trans. Am. Soc. C. E., vol. lviii, p. 458 *et seq.*, Charles E. Gregory concludes that the coefficient C should be taken as a variable, dependent upon the time of concentration t, and offers the expression $C = 0.175t^{1/3}$ for totally impervious areas.

He also suggests for the value of the precipitation factor i, the expression $i = 32/t^{4/5}$. But $t = l/V$, where l = the greatest length of the channel in which water flows from one extremity to the other of the area under consideration, and V = the average velocity of flow in feet per minute. Assuming values for l and V in terms of A and S, the formula reduces to $Q = 2.8\ A^{0.86}S^{0.187}$, which is the Gregory formula as quoted above, for totally impervious surfaces.

Weight Given to the Factors in the Formulas.—For convenience in comparison all the factors except one in the several formulas may be assumed constant, and it is then apparent what weight is given to that factor in each of them. The results of this comparison are expressed in Table 73.

TABLE 73.—EXPONENTS OF THE POWERS TO WHICH THE SEVERAL FACTORS ARE RAISED IN THE VARIOUS FORMULAS FOR RUN-OFF

Formula	Exponent of i (intensity of precipitation)	Exponent of S (slope per 1000)	Exponent of A (area in acres)
Hawksley	0.75	0.25	0.75
Adams	0.833	0.083	0.833
Bürkli-Ziegler	1.00	0.25	0.75
McMath	1.00	0.20	0.80
Hering (A)	1.00	0.27	0.85
Hering (B)	1.00	0.27	0.833
Parmley	1.00	0.25	0.833
Gregory (for impervious surfaces)	1.00	0.186	0.86

THE USE OF McMATH'S FORMULA

Of the foregoing formulas, that of McMath is probably most favorably known, and it has been widely used, often, no doubt, without careful study into its applicability. While we do not recommend the use of

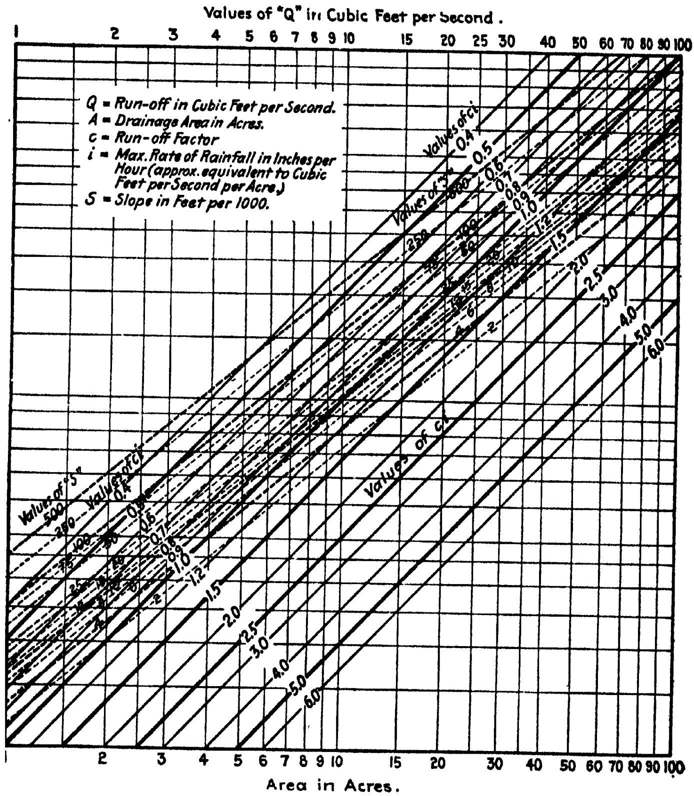


FIG. 104.—Runoff from sewered areas of 1 to 100 acres, by McMath's formula.

this or any similar formula when sufficient information is available for the application of the rational method, yet there are cases when its use may be warranted. It is also convenient for use in rough preliminary computations, as it can be employed very rapidly by means of tables

or diagrams with sufficient accuracy for such purposes, and indeed, with greater precision than the applicability of the formula warrants.

Allen Hazen has prepared tables for the rapid application of McMath's formula, which are contained in the American Civil Engineer's Pocket Book (Second Edition, pp. 969–970), reproduced in Tables 74, 75 and 76. The tables are used as follows:

TABLE 74.—VALUES OF $Ci\sqrt[5]{S}$ IN MCMATH'S FORMULA, TO BE OBTAINED AS A PRELIMINARY TO TAKING THE RUN-OFF FROM THE SUCCEEDING TABLE, BY THE USE OF THE IDENTIFICATION LETTERS

(i taken as 2.75 in. per hour in all cases)

Percentage of total area covered by roofs and pavements		Value of c	Steep slopes 58 per 1000	Average slopes	Flat slopes	Very flat slopes
Sandy soil	Clayey soil					
100	100	0.90	5.58 = A	4.25 = B	3.24 = C	2.47 = D
73	70	0.70	4.25 = B	3.24 = C	2.47 = D	1.89 = E
53	46	0.50	3.24 = C	2.47 = D	1.89 = E	1.44 = F
37	28	0.40	2.47 = D	1.89 = E	1.44 = F	1.10 = G
25	15	0.30	1.89 = E	1.44 = F	1.10 = G	0.84 = H
16	5	0.23	1.44 = F	1.10 = G	0.84 = H	0 64 = I
10	....	0.18	1.10 = G	0.84 = H	0.64 = I	0.49 = J
5	....	0.14	0.84 = H	0.64 = I	0 49 = J	0.37 = K
0	....	0.10	0.64 = I	0.49 = J	0.37 = K	0 28 = L

TABLE 75.—RUN-OFF IN CUBIC FEET PER SECOND PER ACRE, CORRESPONDING TO DATA IN FOREGOING TABLE

Area A in acres	$\sqrt[5]{A}$	Identification letters and corresponding numbers										
		A 5.58	B 4.25	C 3.24	D 2 47	E 1.89	F 1.44	G 1.10	H 0.84	I 0.64	J 0.49	K 0 37
50	2 19	2.55	1.95	1.48	1.13	0.86	0.66	0.50	0.38	0.29	0.22	0.17
70	2.34	2.38	1.82	1.38	1.06	0.81	0.61	0.47	0.36	0.27	0.21	0.16
100	2.51	2.22	1.69	1.29	0.99	0.75	0.57	0.44	0 33	0.25	0.19	0.15
150	2.72	2.05	1.56	1.19	0.91	0.69	0.53	0.40	0.31	0.23	0.18	0.14
200	2.89	1.93	1.47	1.12	0.86	0.65	0.50	0.38	0.29	0.22	0.17	0.13
300	3.13	1.78	1.36	1.04	0.79	0.60	0.46	0.35	0.27	0.20	0.16	0.12
500	3.46	1.61	1.23	0.94	0.71	0.54	0.42	0.32	0.24	0.18	0.14	0.11
700	3.71	1.50	1.15	0.87	0.67	0.51	0.39	0.30	0.23	0.17	0.13	0 10
1,000	3.98	1.40	1.07	0.81	0.62	0 47	0.36	0.28	0.21	0.16	0.12	0.09
1,500	4.32	1.29	0.99	0.75	0.57	0 44	0.33	0.25	0.19	0.15	0.11	0.09
2,000	4.57	1.22	0.93	0.71	0.54	0.41	0.31	0 24	0.18	0.14	0.11	0.08
3,000	4.96	1.12	0.86	0.65	0.50	0.38	0.29	0.22	0.17	0.13	0.10	0.07
5,000	5.49	1.02	0.77	0.59	0.45	0.34	0.26	0.20	0.15	0.12	0.09	0.07
7,000	5.88	0.95	0.73	0.55	0.42	0.32	0.25	0.19	0.14	0.11	0.08	0.06
10,000	6 31	0.88	0.67	0.51	0.39	0.30	0 23	0.17	0.13	0.10	0.08	0 06

To use the tables find in the first place the nearest percentages of area to be covered by roofs and impervious surfaces, under sandy soil or clayey

soil as the case may be, and opposite this in the first table find a letter in the one of the four columns for steep slopes, average slopes, flat slopes and very flat slopes that is selected to represent the conditions. With this letter go to the second table, and follow under it to find a figure opposite the area most nearly equal to the area under consideration. This figure

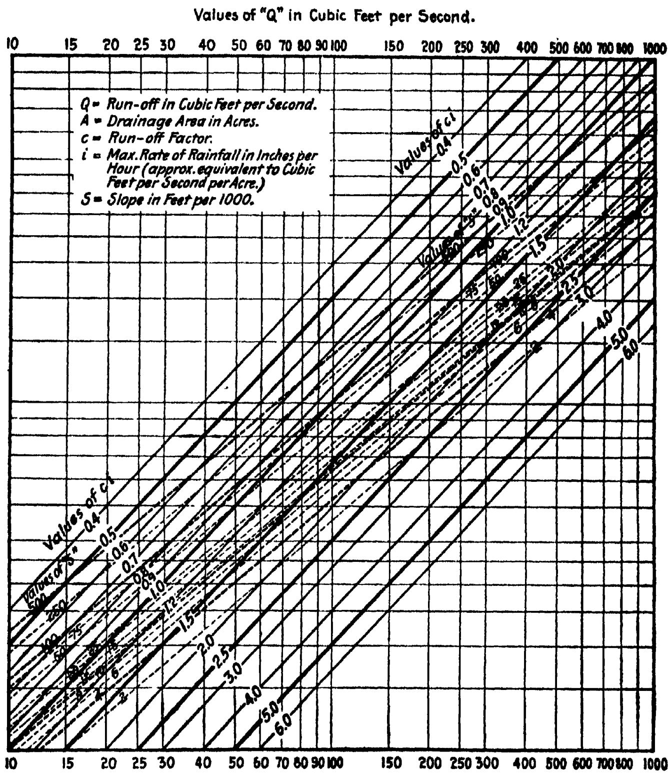


FIG. 105.—Runoff from sewered areas of 10 to 1000 acres, by McMath's formula.

represents the run-off in cubic feet per second per acre that is to be used, and this is to be multiplied by the number of acres. The product is the quantity of storm water in cubic feet per second to be provided for in the sewer. The result is only roughly approximate and is to be accepted with caution.

Convenient diagrams for the solution of McMath's formula are given in Figs. 104, 105 and 106. In using these diagrams, start with the given area at the bottom of the diagram and follow a vertical line to its intersection with the slope line; then follow a horizontal line to its intersection with the ci line, (having first found from Table 77 or by multiplica-

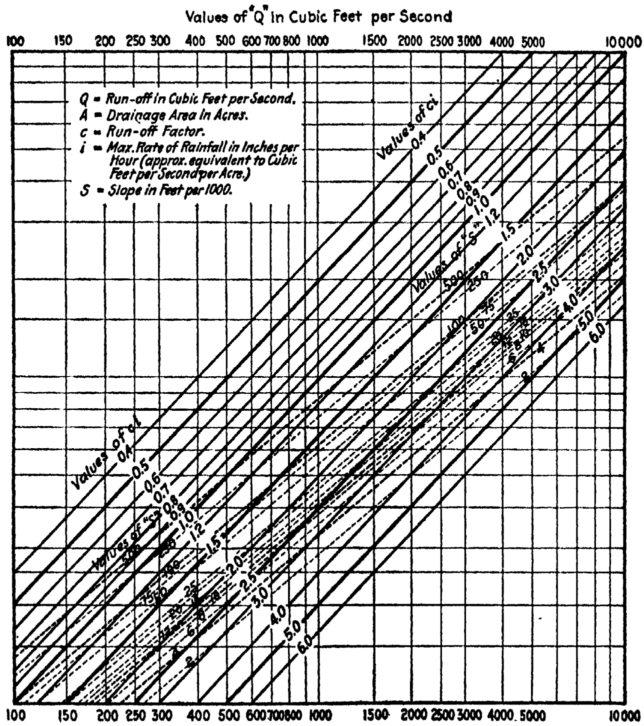


FIG. 106.—Runoff from sewered areas of 100 to 10,000 acres, by McMath's formula.

tion, the product of the assumed coefficient of run-off C and the intensity of precipitation i) from this point follow a vertical line to the scale of quantities at the top of the diagram. For example, assume $A = 100$ acres, $i = 3$ in., $c = 0.70$, and $S = 15$. Then $Q = 144$ c.f.s.

The values of ci for use with these diagrams are given in Table 77.

TABLE 76.—VALUE OF $ci\sqrt[4]{S}$ (IN MCMATH'S FORMULA) USED OR RECOMMENDED FOR VARIOUS CASES. (REARRANGED FROM HAZEN)

Place	Engineer	$ci\sqrt[4]{S}$	
Baltimore........	Kenneth Allen.....	4.25–5.58	....
Washington......	Hering, Gray and Stearns.	4.25±	....
St. Louis.........	McMath..........	3.24±	....
Winnipeg..... ..	....	0.84	Very flat slopes.
Chicago...	....	0.49±	Developed areas 100–10,000 acres.
Chicago..........	....	0.37±	Undeveloped areas.
Watersheds in New England.	Fanning...... .	1.10	Undeveloped areas, 640 acres or over.
Boston (Stony Brook).	Francis, Herschel and Clarke.	1 44	8,000acres,prospects of future development.
Mohawk Valley, natural conditions 10,000 to 100,000 acres, steep slopes, generally impervious soil.	Kuichling...... ..	2.47	Floods which occur occasionally.
		1.48–1.95	Floods which occur rarely.

TABLE 77.—VALUES OF ci FOR USE WITH FIGS. 104, 105, AND 106

c	i					
	2.25	2 50	2.75	3 00	3 50	4.00
0.3	0.68	0.75	0.83	0.90	1.05	1.20
0.4	0.90	1.00	1.10	1.20	1.40	1.60
0.5	1.13	1.25	1.38	1.50	1.75	2.00
0.6	1.35	1.50	1.65	1.80	2.10	2.40
0.7	1.58	1.75	1.93	2.10	2.45	2.80
0.75	1.69	1.88	2.06	2 25	2.63	3.00
0.8	1.80	2.00	2.20	2.40	2.80	3.20
0.9	2.03	2.25	2 48	2.70	3.15	3.60

FLOOD FLOWS FROM LARGE DRAINAGE AREAS

The foregoing empirical formulas have been derived for use in sewer design and are properly applicable only to comparatively small watersheds, seldom exceeding 1000 acres in extent, although they have occasionally been used for much larger areas. It sometimes becomes necessary, in drainage problems, to consider much larger areas, especially in cases where a creek passing through a city is to be converted into a covered channel.

It is a matter of common knowledge that rain-storms cover a somewhat limited area, and it is a fact, though not so generally recognized,

that the more severe the storm, usually the smaller the area that is covered by it. Precipitation of great intensity is usually limited to a very small area and it often happens that the rainfall is not uniform over the whole storm area, but that small districts receive much more precipitation than the area as a whole. Large watersheds may include both steep and gentle slopes, impervious and pervious areas, wooded and arable land, so that portions yielding their run-off rapidly are off-set by others the yield from which is retarded. For these two classes of reasons, the nature of the rainfall and the character of the drainage area, it is obvious that the rate of run-off from a small watershed will be much greater than from a large one.

Many attempts have been made to reduce to formulas the information relating to run-off from watersheds, so that, given the area of the watershed of a stream at any point, the maximum rate of discharge can be computed with reasonable accuracy. A few of these formulas of comparatively recent origin are of interest and are graphically expressed in Fig. 107, in which curves of the McMath and Bürkli-Zeigler sewer formulas are also plotted for comparison.

Kuichling's Formulas.—In the report on the New York State Barge Canal, 1901, Emil Kuichling, after tabulating the various records of run-off and drawing diagrams of all available flood discharge records, prepared two curves "showing the rate of maximum flood discharge on certain American and English rivers, under conditions comparable to those in the Mohawk Valley."

The formula of the first curve gives rates of discharge which may be exceeded occasionally, and is as follows:

$$Q = \frac{44,000}{M + 170} + 20$$

The formula of the second curve gives rates of discharge which may be exceeded rarely, and is

$$Q = \frac{127,000}{M + 370} + 7.4$$

This is for drainage areas of more than 100 sq. miles. For drainage areas less than 100 sq. miles in extent Kuichling has recently suggested the formula (not heretofore published):

$$Q = \frac{35,000}{M + 32} + 10$$

Kuichling has also prepared a formula (not heretofore published), for floods which may be expected to occur frequently. It is

$$Q = \frac{25,000}{M + 125} + 15$$

Kuichling notes that all of these formulas are intended to apply to hilly or mountainous regions, such as are found in the New England,

Middle, and North Atlantic States, and are probably also applicable to a rolling country having a clayey surface soil.

Murphy's Formula.—In Water Supply and Irrigation Paper No. 147 of the U. S. Geological Survey, E. C. Murphy suggests the formula:

$$Q = \frac{46,790}{M + 320} + 15$$

Metcalf and Eddy's Formula.—The authors have suggested[1] the formula (not heretofore published)

$$Q = \frac{440}{M^{0.2688}}, \text{ or } Q = \frac{440}{M^{0.27}}$$

This formula gives results approximating very closely those of the Murphy formula for areas between 100 and 250 sq. miles, and larger results for areas beyond these limits, and is intended to represent floods which may reasonably be expected near Louisville.

In these formulas Q represents the run-off in cubic feet per second per square mile, and M the area of watershed in square miles.

Fuller's Formulas.—The most recent and perhaps most exhaustive study of flood discharge of streams, is contained in a paper on "Flood Flows," by Weston E. Fuller, in "Proceedings" Am. Soc. C. E., May, 1913. Fuller is the first to publish a formula in which the interval of time (corresponding to frequency of floods) appears as a factor in a formula for flood discharge, although it has been recognized for many years that the greater the interval of time, the larger the flood which is likely to occur within that time. It must be remembered that in any such study we deal with averages and probabilities. Because a flood of a certain magnitude is likely to occur once in 100 years, it does not follow that 100 years will elapse before the occurrence of such a flood. If two floods of this magnitude should occur within 5 years, and none thereafter for 195 years, the average occurrence would still be once in 100 years.

The notation used in Fuller's formulas is:

Q = greatest 24-hour rate of run-off in a period of T years, in cubic feet per second,

Q_{max} = the greatest rate of discharge during a maximum flood, in cubic feet per second,

Q_{av} = the average 24-hour flood for a series of years, in cubic feet per second,

T = length of period in years,

M = drainage area in square miles,

C = a coefficient, constant for a given stream at a given point of observation.

[1] In connection with studies for the flood-water discharge of Beargrass Creek, Louisville, Ky.

The formulas derived by Fuller from a study of all available American records are:

$$Q_{av} = CM^{0.8}$$
$$Q = Q_{av}(1 + 0.8 \log T) = CM^{0.8}(1 + 0.8 \log T)$$
$$Q_{max} = Q\left(1 + \frac{2}{M^{0.3}}\right) = CM^{0.8}(1 + 0.8 \log T)\left(1 + \frac{2}{M^{0.3}}\right)$$

In this study it is assumed that the average annual flood flow may be determined with sufficient accuracy from a record extending over a period of 10 to 15 years, in other words, that the average will not be materially affected by increasing the length of the record indefinitely.

Assuming the maximum rate of flood flow (Q_{max}) from a drainage area of 100 sq. miles during a period of 100 years, as unity, the corresponding maximum rates of flood flow for other areas and other periods of time would be as shown in Table 78.

TABLE 78.—RELATION BETWEEN MAXIMUM RATES OF FLOOD FLOW FROM AREAS OF VARIOUS SIZES, AND FOR PERIODS OF VARIOUS LENGTHS, ACCORDING TO FULLER'S FORMULA (WITH A CONSTANT COEFFICIENT)

Drainage area, sq. mi.	Duration of period, in years					
	1	10	50	100	500	1000
	Relative magnitude of maximum flood discharge					
0.1	5.08	9.15	12.0	13.2	16.0	17 2
1.0	1.93	3.48	4.55	5.01	6 09	6.56
5.0	1.04	1.87	2.45	2.70	3.28	3.53
10.0	0.81	1.46	1.91	2.11	2.56	2.76
50.0	0.47	0.85	1.12	1.23	1.49	1.61
100.0	0.38	0.69	0.91	1.00	1.21	1.31
500.0	0.24	0.44	0.57	0.63	0.77	0.82
1,000 0	0.18	0.32	0.46	0.46	0 56	0.60
5,000.0	0 14	0.24	0.23	0.35	0.43	0.46
10,000.0	0.12	0 21	0.27	0.30	0.36	0 39

TABLE 79.—VALUES OF THE COEFFICIENT C IN FULLER'S FORMULA FOR FLOOD FLOWS, FOR VARIOUS SECTIONS OF THE UNITED STATES

Section	No. of drainage areas	Values of C		
		Maximum	Minimum	Average
Atlantic Coast	126	140	30.0	65
St. Lawrence and Upper Mississippi.	39	55	7.5	20
Ohio Basin	38	150	45.0	75
Missouri and Lower Mississippi	74	55	2.0	10
Colorado River	24	45	4.0	15
Pacific Coast	80	210	6.0	40

According to Fuller's studies his formula expresses the general law of variation of flood flows with area and length of period. It is neverthe-

less difficult to select a proper value of C, unless the information available for the stream under consideration is sufficient to enable this to be computed. So many conditions may affect the value of this coefficient that it would be difficult to select a proper value even from the extensive tables given by Fuller. The range in the coefficients computed by him is shown by Table 79.

Fuller does not recommend any value of C for general use when information may not be available for the selection of a coefficient by comparison with some stream for which C has been computed. In his diagram showing a comparison of his formula with other formulas for flood discharge he presents three lines representing his formula, with values of $C = 70$, $T = 100$; $C = 100$, $T = 1000$; and $C = 250$, $T = 1000$, respectively. It may be inferred, perhaps, that a value of $C = 100$ would be reasonable for ordinary use.

With regard to the length of the period to be used, Fuller says:

"Floods have occurred on some rivers during the last 20 years which, normally, would be repeated in not less than 1000 years. If works are to provide for floods equal to the greatest that have been observed, a value of T of at least 1000 should be used. Such a flood or a greater one may occur on any river at any time, but it is not likely to come soon on any particular stream. It must be remembered that the use of $T = 1000$ does not mean that the corresponding flood will come at the end of 1000 years, but that the chances are even that it will occur some time during a period of 1000 years. It means, also, that the chances are 1 to 1000 that it will occur in any one year, or 1 to 100 that it will occur in 10 years, or 1 to 10 that it will occur once in a century. The selection of the proper value of T then becomes a question of what chance we can afford to take."

COMPARISON OF FLOOD FLOW FORMULAS

A comparison of the flood run-offs from drainage areas of various sizes, according to the various formulas for flood discharge, is given in Table 80, and is also shown by the diagram, Fig. 107.

Other Formulas.—Several other formulas have been suggested, and are quoted here merely for reference. It is not believed that any of them is sufficiently applicable to American conditions to be used as a guide in studies of flood discharges, except possibly the culvert formulas, which are probably applicable in cases comparable to those for which they were derived.

In all these formulas

Q = total discharge in cubic feet per second,
M = area of watershed in square miles,
L = extreme length of watershed in miles,
B = average breadth of watershed in miles,
C = a coefficient.

TABLE 80.—COMPARISON OF VARIOUS FORMULAS FOR FLOOD DISCHARGE OF STREAMS, IN CUBIC FEET PER SECOND PER SQUARE MILE

Formula	Drainage area, in square miles							
	1	5	10	50	100	500	1,000	10,000
Kuichling, No. 1 (occasional) $q = \frac{44,000}{M + 170} + 20$	277	272	264	220	183	86	58	24
Kuichling, No. 2 (rare) $q = \frac{127,000}{M + 370} + 7.4$ (for drainage areas of more than 100 sq. miles).	....	....	....	...	277	153	100	19
$q = \frac{35,000}{M + 32} + 10$ (for drainage areas of less than 100 sq. miles).	1070	956	844	437	...	...	.....	...
Kuichling, No. 3 (frequent) $q = \frac{25,000}{M + 125} + 15$	214	207	200	158	126	55	37	17
Murphy (Max. for N. E. U. S.) $q = \frac{46,790}{M + 320} + 15$	161	159	157	141	126	72	51	20
Metcalf and Eddy $q = \frac{440}{M^{0.27}}$	440	286	237	154	127	82	68	37
McMath ($C = 0.75$, $i = 2.75$) $Q = CiA\sqrt[5]{\frac{s}{A}}$ $s = 10$	574	416	362	262	229	165	144	91
Bürkli Ziegler: ($C = 0.9$: $i = 3$) $Q = CiA\sqrt[4]{\frac{s}{A}}$ $s = 10$	611	408	344	230	193	129	109	61
Fuller; $Q_{max} = CM^{0.8}(1 + 0.8 \log T)(1 + \frac{2}{M^{0.3}})$								
$C = 70$, $T = 50$	495	268	209	122	99	62	52	30
$C = 70$, $T = 100$	546	294	230	135	109	69	57	33
$C = 100$, $T = 1000$	1020	550	430	252	204	129	107	61
$C = 250$, $T = 1000$	2550	1375	1070	629	509	322	267	152

Fanning's Formula.—$Q = 200\ M^{5/6}$. This formula was presented by J. T. Fanning in his "Treatise on Water Supply Engineering," 1877. It is based upon a comparatively small number of observations on American streams.

Talbot's Formula.—$Q = 500\ M^{1/4}$. This was intended for use in the prairie states only, and for areas up to 200 sq. miles.

Cooley's Formula.—$Q = CM^{2/3}$, where $C = 180$ to 200.

C. B. & Q. R. R. Culvert Formula.—$Q = 3{,}000\ M/(3 + 2\sqrt{M})$.

Dun's Table.—(For A. T. & S. F. R. R. Culverts.) This gives the following rates of discharge in cubic feet per second per square mile from areas of the given number of square miles.

Area	1	5	10	50	100	500	1000	5000	10,000
Discharge	1,000	910	679	302	212	92	64	27	18

Horner's Culvert Formula.—$Q = 271\ A^{3/4}S^{3/14}/L^{1/4}$, where S is the average slope of the stream, obtained by dividing total fall by length from source to culvert, and A is area in acres. The derivation of this for-

mula is explained in *Engineering News*, May 1, 1913. It is based upon the rational method of storm sewer design, working from the curve of maximum rainfall at St. Louis as derived by Horner, and introducing certain approximations.

Dickens' Formula.—$Q = CM^{3/4}$. This is considerably used by irrigation engineers in India. C may vary between 150 and 1000, and is usually taken as 825.

Ryves' Formula.—$Q = CM^{2/3}$. This formula is also extensively used in India, usually with values of C as follows: within 15 miles of the coast, $C = 450$; from 15 to 100 miles inland. $C = 563$; and for a limited area near the hills, $C = 675$.

Dredge's Formula.—$Q = CM/L^{2/3}$. C is usually taken as 1300. This formula is believed to be based upon studies of rivers in India.

O'Connell's Formula.—

$$Q = -45.796 + \sqrt{2097.28 + (457.96M \times 640)}$$

This formula was proposed in 1868, in a paper contained in Proc. Inst. C. E., vol. xxvii, and is said to have been based on studies of rivers in Europe, India and America.

Craig's Formula.—$Q = 440\,BN$ hyp. log $(8L^2/B)$, where N varies from 0.37 to 1.95, the lower value applying to very flat watersheds. This formula (see Proc. Inst. C. E., vol. lxxx) is intended to apply to Indian conditions.

Ganguillet's Formula.—$Q = 1421M/(3.11 + \sqrt{M})$ for Swiss streams.

Italian Formula.—$Q = CM/(0.311 + \sqrt{M})$, where $C = 1819$ for rivers and 2600 for small brooks in northern Italy.

Possenti's Formula.—$Q = \dfrac{CR}{L}\left(M_1 + \dfrac{M_2}{3}\right)$ when C has an average value of 1010, and R is depth of rain in inches per 24 hours, M_1 is the area of the hilly or mountainous, and M_2 the area of the flat portion of the watershed.

Cramer's Formula.—$Q = \dfrac{CR'mMS^{1/3}}{9 + (0.0658mR'M)^{1/3}}$ in which C varies from 186 for rough, natural drainage areas, to 698 for smooth, comparatively level and impervious areas, such as may occur in cities.

R' = mean annual rainfall in inches,

S = average slope of stream from source to point of observation,

m is a factor depending on M, R', and the flat area F subject to overflow.

When this is distributed in a uniform manner throughout the basin

$$m = 1 - \sin\left(\tan^{-1}\frac{709F}{MR'}\right)$$

and if the flat area is concentrated at the lowest point, then

$$m = 1 - \sin\left(\tan^{-1}\frac{1418F}{MR'}\right)$$

Lauterburg's Formula.—$Q = M\left(\frac{615}{6 + 0.00259M} + 0.53\right)$ intended to apply to floods resulting from a continuous heavy rain of 3 or 4 days duration at an average rate of 2 in. per day.

FLOODS

Effect of Snow and Ice.—It may happen in some cases that the maximum flow of streams will occur when a warm rain falls upon snow already on the ground, or when the ground may be coated with ice in such a manner as to present a practically impervious surface, as well as allowing a portion of it to melt and run off with the rain. In these cases the total run-off may amount to 100 per cent. of the precipitation, or even more. In the case of streams of considerable magnitude, where the time necessary for concentration is several hours, or possibly even days, and where the maximum rate of precipitation, which probably prevailed over but a limited area, is a comparatively small factor in determining the maximum rate of run-off, maximum flood conditions are particularly likely to occur from rain falling upon snow or ice.

In such cases it is desirable to estimate the approximate equivalent of the snow or ice, upon the ground, in terms of depth of water. The United States Weather Bureau "Instructions to Co-operative Observers" state that when it is impossible to measure the water equivalent of snow by melting, one-tenth of the measured depth of snow on a level open place is to be taken as the water equivalent, although it is recognized that this relation varies widely in different cases, depending on the wetness of the snow. The water equivalent of snow may be as great as one-seventh or as small as one-thirty-fourth of the depth of the snow. These figures apply to recently fallen snow; the water equivalent of snow which has been on the ground for some time and which is therefore compacted to some extent, would be greater. R. E. Horton states in the "Monthly Weather Review," May, 1905:

> All records indicate that for the heavy and persistent snow accumulations occurring in New York and New England a progressive growth in the water equivalent per inch of snow on ground will usually take place as the season advances, due to compacting by wind, rain and partial melting, and to the weight of the superincumbent mass on the lower layers. The water equivalent of compacted snow accumulation is commonly between one-third and one-fifth, or at least double that for freshly fallen snow.

The relation between the thickness of an ice layer and the corresponding depth of water is more uniform, and for practical purposes 1 in. of ice may be considered as equivalent to 0.9 in. of rain.

In the case of sewer districts, maximum run-off is much less likely to occur from rain falling upon snow or ice. Rains of great intensity are of comparatively rare occurrence during the season when snow or ice

are formed. Moreover, the effect of snow upon the ground would usually be to retard the flow of water, the snow acting as a sponge during the time of heaviest precipitation, and causing the run-off to be at a more gradual rate than the rainfall during this portion of the storm. It is, however, possible, under extreme conditions, that maximum run-off might be caused by a warm rain of heavy intensity following after a period of comparatively light precipitation, by which the snow has been saturated and nearly melted, so that the maximum rate of run-off might even be in excess of the greatest rate of precipitation, and the possibility of this condition must always be borne in mind.

Records of Flood Flow of Streams.—Table 81 contains some records of flood flow of streams in the United States.[1]

TABLE 81.—DRAINAGE AREA AND MAXIMUM DISCHARGE FOR VARIOUS AMERICAN RIVERS

Name of stream and locality	Drainage area, sq. mi.	Max. dis. cu. ft. per sec. per sq. mi	Date	Authority
Budlong Creek, Utica, N. Y.	1.13	120.40	1904	U. S. Geol. Sur., W. S. P. No. 147
Sylvan Glen Creek, New Hartford, N. Y.	1.18	56 58	1904	U. S. Geol. Sur., W. S. P. No. 147
		277.00	......	W. E. Fuller. Trans. Am Soc. C E vol. lxxvii.
Pequest River, Hunts Pond, N. J.	1.70	25.30	......	N. J. Geol. Sur., 1894 Pt. 4
Starch Factory Creek, New Hartford, N. Y.	3 40	109.62	1904	U. S. Geol. Sur., W. S. P. No. 147
		209.00	1905	U. S. Geol. Sur., W. S. P. No. 162
Reels Creek, Deerfield, N. Y.	4.40	48.36	1904	U. S. Geol. Sur., W. S. P. No. 162
Mad Brook, Sherburne, N. Y.	5.00	262.00	1905	U. S. Geol. Sur., W. S. P. No. 162
Skinner Creek, Mannsville, N. Y.	6.40	124.20	1891	U. S. B. Engrs. D. W., 1899
Coldspring Brook, Ashland, Mass.	6.43	48.40	1886	Trans. Am. S. C. E., Vol. 25
Croton River, So. Branch, N. Y. (30-yr. record).	7.80	73.90	1869	Trans. Am. S. C. E., Vol. 4
Mill Brook, Edmeston, N. Y.	9.40	241.00	1905	U. S. G. S., W. S. P. No. 162
Woodhull Reservoir, Herkimer, N. Y.	9.40	77.80	1869	Trans. Am. Soc. C. E., Vol. 4
Stony Brook, Boston, Mass.	12.70	121.00	1886	Rept. Stony Br. Flood Com.
Manhan River, Holyoke, Mass.	13.00	182.00	1900	James L. Tighe by letter
Great River, Westfield, Mass.	14.00	71.40[a]	......	Rept. of H. F. Mills
Swartswood Lake, N. J.....	16.00	68.00	......	N. J. Geol. Sur., 1894 Pt. 4.
Williamstown Riv. at upper dam, Williamstown, Mass.	16.20	30.90	......	U. S. B. Eng. D. W., 1899
Williamstown Riv. at lower dam, Williamstown, Mass.	16.50	34.00	......	....
Croton River, W. Branch, N. Y. (30-yr. record).	20.47	54.40	1874	E. M. Treman, J. J. R. Croes, Tech. Quar. 1891, p. 325.
Beaver Dam Creek, Altmar, N. Y.	20 70	111.00	......	U. S. B. Engrs. D. W., 1899.

[1] See also Trans. Am. Soc. C. E., vol. lxxvii, pp. 650-658.

[a] Average flow for day of maximum discharge.

TABLE 81.—DRAINAGE AREA AND MAXIMUM DISCHARGE FOR VARIOUS AMERICAN RIVERS.—*Continued*

Name of stream and locality	Drainage area, sq. mi.	Max. dis. cu. ft. per sec. per sq. mi.	Date	Authority
Trout Brook, Centerville, N. Y.	23.00	50.6	1875	U. S. B. Engrs. D. W., 1899.
Pequonnock River, Bridgeport, Conn.	25.00	157.00	1905	Water Supply Paper 162.
Watuppa Lake, Fall River, Mass.	28.50	69.70	1875	Rep. N. Y. Barge Canal, 1901 Trans. Am. S. C. E., Vol. 4
Watuppa Lake, Fall River, Mass.	28.50	72.00	1875	Trans Am. S. C. E., Vol. 4
Pequest River, Huntsville, N. J.	31.40	19 30	.....	N. J, Geol. Surv,, 1894 Pt, 3
Pequest River, Tranquillity, N. J.	34.80	18.70	.....	....
Sawkill River, Kingston, N, Y,	35.00	228.60	1896	U, S, Geol, Sur., W, S, P. No. 35.
Whippany River, Whippany, N. J.	38.00 37.00	84.20 61.62	1896 1903	U. S. Geol. Sur.
Cayadutta Creek, Johnstown, N. Y.	40.00	72.40	1896	U. S. B. Engrs. D W., 1899
Six-mile Creek, Ithaca, N.Y.	46.00	132.00 195.00	 1905	Emil Kuichling U. S. Geol. Sur., W. S. P. No. 162
Mad River at Camden, N. Y.	46.60	22.10	......	Rep. N. Y. Barge Canal, 1901 (U. S. B. Engrs., D. W. 1899.)
W. Canada Creek, Motts Dam, N. Y.	47.50	34.10	1894	U. S. B. Engrs. D. W., 1899
Little Conemaugh, So. Fork, Johnstown, Pa.	48.60	205.70	1889	Trans. Am. Soc. C. E., Vol. 24, 1891.
Sauquoit Creek, N. Y. Mills, N. Y.	51.50	53.40	......	U. S. B. Engrs. D. W., 1899
Rockaway River, Dover, N. J.	52 50	43.00	......	N. J. Geol. Surv., 1894
Mill River, Mass..........	58.00	15.50	......	Rep. N. Y. Barge Canal, 1901
Oneida Creek, Kenwood, N. Y.	59.00	41.20	1890	U. S. B. Engrs. D. W., 1899
Flat River, R. I...........	61.00	120.90	1845	Trans. Am. S. C. E., Vol. 4
Camden Creek, Camden, N. Y.	61.40	24.10	1889	U. S. B. Engrs. D. W., 1899
Pequonnock River, Macopin, N. J.	62.00	90.80	1903	....
Nine Mile Creek, Stittville, N. Y.	62.60	124.90	1898	U. S. B. Engrs. D. W., 1899
Otter Creek, N. Y., Castor's Mills.	63.00	30.90	1869	Rept. N. Y. Barge Canal, 1901
Wissahickon Creek, Philadelphia, Pa.	64.60	43.50	1897 1898	U. S. Geol. Surv. 20th An. Rept.
Musconetcong Creek, Saxton Falls, N. J.	68.00	15.90	......	Rept. N. J. Geol. Surv., 1894 Pt. 3
Kinderhook Cr., Garfield, N. Y.	68.20	9.00	1894	....
Sandy Creek, So. Branch, Allendale, N. Y.	68.40	87.70	1890 1891	U. S. B. Engrs. D. W., 1899
Sudbury River, Framingham, Mass.	74.65	44.30	1886	Trans. Am. S. C. E., Vol. 25

TABLE 81.—DRAINAGE AREA AND MAXIMUM DISCHARGE FOR VARIOUS AMERICAN RIVERS.—*Continued*

Name of stream and locality	Drainage Area, sq. mi	Max. dis. cu. ft. per sec. per sq. mi.	Date	Authority
Park River, Conn.........	76.00	30.40	......	....
Rock Creek, Georgetown, D. C.	77.50	126.30	......	Tech. Quar. M. I. T., 1891, p. 242. Trans. Am. S. C. E., Vol. 15.
Sudbury River, Framingham, Mass.	78.00	41.38	1897	Eng. Water Dept. City of Boston.
Pequonnock River, Pompton, N. J.	78.00	55.78	1902	U. S. Geol. Surv., unpublished
Hockanum River, Conn ...	79.00	78.10	......	Ch. Engr. U. S. A., 1883.
Garoga Creek, 4 mi. above Ft. Plain, N. Y.	80.80	15.80	......	....
Pequest River at Townsburg, N. J.	83.40	9.60	......	....
Nashua River, Mass...	84.50	71.04	1850	Trans. Am. S. C. E., Vol. 4
Pequonnock River, Riverdale, N. J.	84.70	52.50	1882	Rept. N. Y. Barge Canal, 1901,. Geol. Sur. N. J. 1894
Roach River, Roach River, Maine.	85.00	23.2	......	1st An. Rep Me. St. W. Stor. Com. 1910, p. 350.
Independence Creek, Crandall's Mills, N. Y.	93.20	66.50	1869	....
Passaic River at Chatham, N. J.	100.00	17.20	1903	U. S. Geol. Surv. (unpublished)
Tohickon, N. J.	102.00	{ 112.5	1885	....
		138.00	. ....	W. E. Fuller.
Nashua River, Mass.......	109.00	104.6	......	Tr. Am. S. C. E., Vol. 4
Pacolet River, Mahwah, N. J.	118.00	105.1	......	W. S. and I, papers 147, p. 185
Scantic River, Conn........	118.00	51 8	......	Rep. Ch. Eng., U. S. A., 1878.
Rockaway River, Boonton, N. J.	121.00	62.5	1903	....
Wanaque River, Pompton, N. J.	162.00	65.9	1882	....
Pawtucket River, R. I......	190.00	56 9	1867	Tr. Am. S. C. E., Vol. 4
Cobboseecontee River, Gardiner, Me. (21-yr. record).	240.00	13.6	1903	1st An. Rep. Me. St. W. St. Com., 1910, p. 350.
Piscataquis River, Foxcroft, Me. (9-yr. record).	286.00	77.6	1909	1st An. Rep. Me. St. W. St. Com., 1910, p. 350.
Sebasticook River, Pittsfield, Me. (3-yr. record).	314.00	22.8	1909	1st An. Rep. Me. St. W. St. Com., 1910, p. 350.
Croton River, N. Y........	338.82	74.9	1854	Tech. Quar., 1891.
Carrabassett River, No. Anson, Me. (7-yr. record).	340.00	40.3	1904	1st An. Rep. Me. St. W. St. Com., 1910, p. 350.
Westfield[1] River, Salmon Falls, Mass.	350 00	151.4	1878	Rept. of H. F. Mills
Saco River, Center Conway, N. H. (8-yr. record).	385.00	36.7	1907	1st An. Rep. Me. St. W. St Com., 1910, p. 350
Presumpscott River, Sebago Lake, Me. (24-yr. record).	420.00	33.0	1896	1st An. Rep. Me. St. W. St. Com., 1910, p. 350.
Machias River, Whitneyville, Me. (8-yr. record).	465.00	23.9	1909	1st An. Rep. Me. St. W. St. Com., 1910, p. 350.
Olentangy River, Columbus O.	520.00	115.0	1913	Alvord and Burdick.

[1] Rain 3 in., melted snow 6 to 8 in.

TABLE 81.—DRAINAGE AREA AND MAXIMUM DISCHARGE FOR VARIOUS AMERICAN RIVERS.—*Continued*

Name of stream and locality	Drainage area, sq. mi.	Max. dis. cu. ft. per sec. per sq. mi.	Date	Authority
Sandy River, Madison Me. (5-yr. record).	650	21.3	1907	1st An. Rep. Me. St. W. St. Com., 1910, p. 350
Moose River, Rockwood, Me.	680	10.0	1908	1st An. Rep. Me. St. W. St. Com., 1910, p. 350.
Dead River, The Forks, Me.	878	21.0	1907	1st An. Rep. Me. St. W. St. Com., 1910, p. 350.
Fish River, Wallagrass, Me. (6-yr. record).	890	10.1	1908	1st An. Rep. Me. St. W. St. Com., 1910, p. 350.
Penobscot River, E. Branch, Grindstone, Me. (9-yr. record).	1,100	23.4	1909	1st An. Rep. Me. St. W. St. Com., 1910, p. 350.
St. Croix River, Woodland, Me. (9-yr. record).	1,420	14.3	1909	1st An. Rep. Me. St. W. St. Com., 1910, p. 350.
Mattawamkeag River, Mattawamkeag, Me. (9-yr. record).	1,500	16.3	1907	1st An. Rep. Me. St. W. St. Com., 1910, p. 350.
Saco River, West Buxton, Me. (4-yr. record).	1,550	13.4	1909	1st An. Rep. Me St. W. St. Com., 1910, p. 350.
Kennebec River, The Forks Me. (10-yr. record).	1,570	11.7	1902	1st An. Rep. Me. St. W. St Com., 1910, p. 350.
Scioto River, Columbus, O.	1,570	89.0	1913	Alvord and Burdick.
Penobscot River, West Branch, Millinocket, Me. (10-yr. record).	1,880	12 9	1903	1st An. Rep. Me. St. W. St. Com., 1910, p. 350.
Androscoggin River, Rumford Falls, Me. (19-yr record).	2,090	26.4	1895	1st An. Rep. Me. St. W. St. Com., 1910, p. 350.
Aroostock River, Fort Fairfield, Me. (8-yr. record).	2,230	15.4	1907	1st An. Rep. Me. St W. St Com., 1910, p. 350
Kennebec River, Bingham, Me. (4-yr. record).	2,660	11.7	1909	1st An. Rep. Me. St. W. St. Com., 1910, p. 350
Kennebec River, No. Anson, Me. (7-yr. record).	2,790	13.5	1907	1st An. Rep. Me. St. W. St. Com., 1910, p. 350.
Androscoggin River, Lewiston, Me. (61-yr. record).	2,950	22.1	1896	1st An. Rep. Me. St. W. St. Com., 1910, p. 350.
Kennebec River, Waterville, Me. (18-yr. record).	4,270	35.7	1901	1st. An. Rep. Me. St. W. St. Com., 1910, p. 350.
Hudson River, Mechanicsville, N. Y. (26-yr. record).	4,500	25.2	1913	Horton, Bulletin Z., U. S. Weather Bureau.
St. John River, Fort Kent, Me. (6-yr. record).	5,280	14.3	1909	1st. An. Rep. Me. St W. St. Com. 1910, p. 350.
Penobscot River, West Enfield, Me. (9-yr. record).	6,600	14.6	1909	1st An. Rep. Me. St. W. St. Com. 1910, p. 350
Penobscot River, Bangor, Me. (10-yr. record).	7,700	15.0	1901	1st An. Rep. Me. St. W. St. Com. p. 350.

Frequency of Flood in Streams.—An elaborate study of the relative magnitude of flood flows to be expected in various periods of time is contained in Weston E. Fuller's paper on "Flood Flows," previously referred to. According to his analysis, the greatest flood which is likely to occur in a period of T years will exceed the average annual flood by

0.8 log T times the average annual flood. The relative magnitudes of floods which will probably occur in periods of various durations according to this relation, are shown in Table 78.

Robert E. Horton, in Bulletin Z of the U. S. Weather Bureau, on The Floods of 1913, has explained the application of the mathematical theory of probabilities to the estimation of the probable recurrence of floods of various magnitudes, and derived the formula

$$T = \left(\frac{Q + 50{,}000}{80{,}000}\right)^{7.14}$$

for the Hudson River at Mechanicsville, where the drainage area is 4500 sq. miles. In this formula T = average period of recurrence in years and Q = maximum flood flow in cubic feet per second. In a discussion upon Fuller's paper on "Flood Flows" Horton also gives[1] the general formula

$$T = \sqrt[4]{\frac{QM}{4021.5}}$$

(where M = drainage area in square miles) derived from 20-year records of Neshaminy, Perkiomen and Tohickon Creeks, near Philadelphia.

Information indicating the relative frequency of floods of various magnitudes on twenty-three American rivers are given by E. C. Murphy in Water Supply and Irrigation Paper No. 162 of the U. S. Geological Survey. The most significant information, compiled from his records, is given in Table 82.

Design of Flood-water Channels.—One of the best studies of flood discharge of streams is contained in the classic report on "Prevention of Floods in the Valley of Stony Brook" (Boston), by James B. Francis, Eliot C. Clarke, and Clemens Herschel. This report was made in 1886, following a destructive flood in February of that year, when over 1400 buildings were affected. The total watershed of this brook was 13.92 sq. miles, and the storm from which the flood resulted included 5.86 in. of rain; snow and ice on the ground were estimated to correspond to about 2 in. more, making an equivalent of about 8 in. of rain in 3 days and 7 hours. After an exhaustive study, the engineers concluded that none of the run-off formulas discussed by them (the Dickens, Dredge, O'Connell, Craig, Fanning, and Bürkli-Ziegler formulas) was pertinent in this case; that a rainfall of 12 in. in 24 hours was to be expected; that it was not probable that this would run off at a rate greater than 0.25 of the rate of precipitation in that time, but that ultimately, when the region became densely built up, the rate of run-off might reach 0.75 of the rate of precipitation. This, however, would probably be so far in the future that 0.50 of the rate of precipitation represented as large a capacity as should be given the flood channels.

[1] Trans. Am. Soc. C. E., vol. lxxvii, p. 665.

TABLE 82.—AVERAGE INTERVAL BETWEEN FLOODS OF VARIOUS MAGNITUDES IN SOME AMERICAN RIVERS

River	Area of watershed, sq. mi.	Length of record, years	Maximum observed flood, c.f.s. per sq. mi.	Magnitude of flood as compared to maximum flood			
				0.6 to 1.0	0.7 to 1.0	0.8 to 1.0	0.9 to 1.0
				Average frequency, years			
Kennebec......	4,380	12	25.4	2	4	12	12
Androscoggin...	2,320	12	23.8	4	6	12	12
Merrimac......	4,553	59	18.0	......	....	15	20
Connecticut....	10,234	105	20.0	......	....	12	53
Hudson........	4,500	35	15.6	......	....	18	35
Genesee........	2,428	119	19–22	......	....	60	119
Passaic........	8,227	26	42.5	......	13	26	26
Raritan........	806	96	64.5	.....	24	48	96
Delaware.......	6,855	120	37.1	....	......	40	120
Susquehanna ..	24,030	17	28–30.6	.....	6	9	17
Cape Fear......	3,860	15	18–23	0.4	0.6	2	7
Savannah......	7,500	66	40	0.8	1.5	5.5	33
Alabama.......	15,400	14	9.5	0.6	0.7	1.6	5
Black Warrior...	4,900	17	32	0.4	0.5	0.9	2
Monongahela...	5,430	20	38.1	5	10	20	20
Youghiogheny...	782	32	54–59	4	16	32	32
Allegheny.....	9,220	31	26.7	0.3	1	3	8
Ohio..........	23,800	22	20.8	2	3	10	22
Illinois...	15,700	16	3.3	......	2	3	5
Rio Grande....	28,067	11	1.2	4	6	6	11
Colorado.......	37,000	9	3.3	.....	5	9	9
Arkansas.......	4,600	10	2.4	3	5	10	10
Bear..........	6,000	15	1 8	3	5	15	15

Another careful study of the design of a storm-water channel is contained in the "Special Report to the Commissioners of Sewerage of Louisville upon the Improvement of Beargrass Creek," by J. B. F. Breed and Harrison P. Eddy, in 1909. This stream drains a watershed of 65.4 sq. miles, including the easterly portion of the city of Louisville. A detailed study of existing data relating to flows of this stream and the precipitation in Louisville was made, and compared with records of flood flows of streams in the northeastern United States, and with the Kuichling and Murphy formulas. It was concluded that the Murphy formula gave too small results for this locality, and that provision ought to be made for flood discharges amounting to about 180 cu. ft. per second per square mile.

CHAPTER VIII

THE RATIONAL METHOD OF ESTIMATING STORM-WATER RUN-OFF IN SEWER DESIGN

Few problems have afforded the sewer designer more misgivings than the determination of the quantity of storm water for which storm drains or combined sewers should provide. The chief reason for this lies in the fact that the problem is indeterminate, and that the information which may be available and the formulas which may be used only serve to aid his judgment, upon the soundness of which the correctness of final solution very largely depends. In fact, it is a difficult task to say when the solution of such a problem is correct within the usual meaning of the term, because no two engineers acting independently would be likely to reach the same conclusions as to the economic period in the future upon which to base the design of the system, the ultimate development and improvement of the district within this economic period, the rate of rainfall for which the community can reasonably be expected to provide drainage, and the rate at which the storm water will reach the sewers, all considerations vitally affecting the sizes of the drains or sewers being designed.

The earliest attempts to solve this problem were based upon observations or estimates of flow in existing streams, gutters and drains. Formulas of an empirical character were derived from such studies, which have been quoted and described in the preceding chapter. Finally, the attention of engineers has been focused upon the fact that the run-off is directly dependent upon the rate of rainfall and the rapidity with which the water will reach the drains. This is a long step in advance, but the problem is still quite indeterminate and requires for its economic solution sound judgment aided by experience and by all the information which can be obtained.

Conditions Affecting Rate of Run-off.—The volume of storm water to be cared for by a sewer or drain depends upon the intensity and duration of the rain, and the character, slope and area of the surface upon which it falls. Of the water falling upon the surface, a portion is lost by evaporation; still another is required to fill the depressions of the surface; another portion sinks into the earth, where it is either retained by capillary attraction or else percolates slowly through the earth to reinforce the ground water and to reappear at some lower point in springs

or streams; another portion is absorbed by vegetation; while the remainder flows off over the surface until collected in natural or artificial channels. This last portion is the one with which the problem of storm drainage is concerned.

The proportion of the total rainfall which will flow off from any given area varies with the duration and intensity of the rain and with the amount of moisture in the earth before the storm, and also with the condition of the surface of the ground, whether frozen or covered with snow or ice. It will also change from time to time on the same area as the character of the surface is artificially modified by the construction of streets, pavements, and buildings.

It is evident that the run-off from any given area will be greatest when all parts of the area are contributing at the greatest possible rate. This requires a lapse of time, not only to allow the water flowing from the most distant part of the area to reach the outlet, but also to fill depressions and saturate the surface soil. The maximum run-off is therefore to be expected from a rainfall of maximum uniform intensity lasting as long as the period of time required to allow the water from the farthest point of the drainage area to reach the outlet; but on the other hand the maximum flow during many storms occurs when some portions of the district are contributing water at a much smaller rate than other portions, because of wide fluctuation in the intensity of the precipitation upon different portions of the tributary area.

At the present time the so-called rational method of estimating the amount of run-off is commonly employed in the design of storm-water or combined sewers. Even in St. Louis, the home of the McMath formula, that formula has been displaced and the "rational method" is now used in sewer design.

The rational method recognizes as axiomatic the direct relation between the rainfall and the run-off, as shown by the formula $Q = CiA$, in which Q = the total amount of run-off from a given area in cubic feet per second; C = a coefficient representing the ratio of run-off to rainfall, generally called the run-off coefficient or the coefficient of imperviousness; i = the intensity of rainfall in cubic feet per second per acre (or nearly enough, the rate of rainfall in inches per hour); A = the drainage area in acres.

In a computation by this method, the area A is definitely determined by measurement. It is also necessary to determine, first, the time of concentration, which is the length of time required for the water to flow from the most distant point of the district to the nearest sewer inlet, and thence through the sewers to the point of observation; second, the greatest uniform intensity of rainfall corresponding to this period of time, or, at least, the greatest intensity for which provision should be made in the design of sewers; and third, the run-off coefficient or coefficient of imper-

viousness, which depends upon the character of the soil, slope and character of the surface.

Time Required for Water to Reach the Sewers (Inlet Time).—The time required for flow over the surface and into the sewer must either be estimated from the available information or be determined by observation. It will seldom be less than 3 or more than 20 minutes. In the case of small districts, or fairly large districts with steep slopes, this time is frequently the most important element in determining the quantity of water for which to provide. W. W. Horner states (*Eng. News*, Sept. 29, 1910) that he has reached the conclusion, based upon actual observations, that the water from the streets and sidewalks and roofs will reach the sewer in from 2 to 5 minutes, with street grades of from 1/2 to 5 per cent. (improved streets), but that the velocity over grass plots is very low, and even in heavy rains from 10 to 20 minutes will be required for the water to flow 100 ft. For the sake of safety a short time should be assumed, and allowance made for lawns and grass plots by assuming a suitable coefficient of run-off.

Charles E. Gregory, in his discussion of Grunsky's paper upon "The Sewer System of San Francisco," has computed theoretically (*Trans. Am. Soc. C. E.*, vol. lxv, p. 393) the rate of run-off in a gutter 1000 ft. long, having a slope of 0.0025, draining an impervious street surface 24 ft. wide, when there is a uniform rainfall at the rate of 4 in. per hour, and finds that this rate of precipitation would have to continue for 42 minutes before the rate of discharge would equal the rate of precipitation, and that 25 minutes would elapse before the rate of run-off equalled half the rate of precipitation. His conclusion is that for many roofs and a few street surfaces, where the storm-water inlets are moderately closely spaced, the common assumption of 5 minutes as the time required for the storm water to reach the sewer at maximum rate may be true, but in most cases this time is materially greater, and that it varies widely under different circumstances.

In view of the lack of any definite information relating to individual sewer districts, the following information relating to the run-off from an area of 0.055 acre in a small city in Arkansas where the soil was heavy and sun-baked, but without any paved or roof surfaces, is significant. This information was presented by James H. Fuertes in a discussion in *Jour. West. Soc. Engs.*, April, 1899, p. 170. He says:

"Several years ago the opportunity was presented of measuring the run-off from a small tract of ground in a southern city. Although the observations were made with hastily improvised apparatus and the tract of ground was quite small, the writer offers it with suitable apologies for its meagerness, because of the scarcity of published records of such information for either large or small tracts. The tract of ground sloped quite uniformly in two directions toward one corner, the fall of the surface being about 5 ft.

in 100 ft. Along one side a ditch was cut, into which the water drained from the whole area. At the end of the ditch a small weir was arranged, and the depth of the water flowing over the weir was measured with a thin ivory scale at as frequent intervals as the observations could be recorded: varying from a minute to about 3 minutes. The rain depths were similarly measured, though at less frequent intervals. The total depth of rain that fell upon the tract, in the particular storm in question, was 1.3 in. which fell in 37 minutes. The maximum rate of rainfall was 6 in. per hour, which continued about 5 minutes and was reached 11 minutes after the beginning of the storm.

"At the beginning of the storm the ground was very hard and dry. The tract was a heavy, clayey soil, covered with a short and rather thin growth of grass. From the data obtained, it was deduced that 29 per cent. of the total rainfall on the tract passed over the measuring weir; that the average velocity of the water in the ditch was about 4 ft. per second; and that the average velocity of the water flowing over the surface of the ground to the ditch was about 0.1 ft. per second."

The diagram accompanying this discussion shows that rain began at 6.40, and run-off at the gaging point at 6.47; maximum rainfall rate began at 6.51, and maximum rate of run-off was attained at 6:59; from which it may be deduced that the time of concentration for this area, which would be the inlet time if this district were tributary to a single sewer inlet, was about 8 minutes.

The rain continued at the maximum rate of 6.0 in. per hour for but 5 minutes. The average rate of precipitation for the 8 minutes of greatest rainfall was about 5.3 in. per hour, and the maximum run-off was 7.2 cu. ft. per minute, equivalent to 2.18 cu. ft. per second per acre, or 41 per cent. of the rainfall rate for 8 minutes. The run-off factor was therefore 0.41.

Time of Concentration.—As defined above, the time of concentration is the time required for the water to flow from the most distant point (measured in time) to the point under consideration. It is made up of two parts, the inlet time and the time of flow in the sewers. Inlet time has been discussed in the preceding section. The time of flow in the sewers is readily obtained by a simple hydraulic computation if the conditions, quantity of water and size and slope of sewers are known.

It is important to distinguish the minimum time of concentration from what may be called the actual time of concentration. The former corresponds to the conditions for which sewers should be designed; conduits full (or half full), and velocity substantially at a maximum, and conditions of surface such that run-off from roofs and streets and flow in gutters will be at maximum rates. Under these conditions the time of concentration will be a minimum and the corresponding rate of precipitation will be a maximum. The conditions are therefore the most serious to which the sewer may be subjected. The minimum

time of concentration is a constant for a given sewer district in a particular state of development.

On the other hand, the actual time of concentration represents the time required for the concentration of the waters of a particular storm, under the conditions existing at the moment. If the storm is of moderate intensity, the sewer may be but partly filled and the velocity of flow may therefore be considerably less than the maximum. Moreover, unless rain has previously been falling for some time, the filling of depressions and the accumulation of sufficient head to cause flow over rough or nearly flat surfaces will require an appreciable amount of time. The actual time of concentration will therefore exceed the minimum in all cases except those for which the sewer was designed.

In problems of sewer design the engineer is concerned only with the minimum time of concentration; but when gagings of storm-water flow are made it is important to recognize that the conditions are reversed, and it is then the actual time of concentration which represents the period of rainfall with which the resulting flow must be compared.

RUN-OFF FACTOR

The coefficient of run-off is very difficult of exact determination, even for existing conditions, and is subject to great modification by artificial alterations in the conditions of the surface, such as changes in the extent of the built-up district and in areas covered by paved streets. It is therefore necessary in designing sewers to estimate the conditions which are likely to obtain a reasonable time in the future, unless the district under consideration has already reached such a degree of development that no further changes are probable.

The run-off factor or coefficient, sometimes called the coefficient of imperviousness, depends upon a large number of elements and is not constant for a given area, even during a single storm. It was formerly considered that this factor represented strictly the actual percentage of impervious surface in the district under consideration, and that if the entire surface were covered with impervious materials, such as roofs and asphalt pavements, the factor would be 1.00. More recently, however, it has developed that the factor is seldom unity, even for an absolutely impervious surface. Some evaporation always takes place, even during the progress of a rain storm, and even the most impervious surfaces absorb small quantities of water. Irregularities of the surface also tend to hold back some of the water and prevent its running off as rapidly as it falls.

In this connection it may aid the engineer in forming a conception of the problem of run-off to consider the quantity of water actually falling in several periods of time, as given in Table 83, computed from the curve, $i = 15/t^{0.5}$.

TABLE 83.—QUANTITY IN INCHES OF RAIN FALLING IN THE SPECIFIED PERIODS OF TIME AT THE RATES INDICATED BY CURVE OF INTENSITIES, $i = 15/t^{0.5}$

Time, minutes	Rate of precipitation, inches per hour	Accumulated depth of precipitation, inches
5	6.71	0.56
10	4.75	0.79
15	3.88	0.97
20	3.36	1.12
30	2.75	1.38
45	2.24	1.68
60	1.94	1.94
90	1.58	2.37
120	1.37	2.74

Note that these periods of time are not necessarily measured from the beginning of a storm, or even from the beginning of the downpour. Prof. A. J. Henry of the U. S. Weather Bureau gives (in Bulletin D, Rainfall of the United States, and also in *Jour. West. Soc. Engs.*, April, 1899) a table showing the percentage of cases of downpour in Washington, Savannah, and St. Louis, in which the maximum rate of precipitation occurred at various periods after the beginning of the storm. This information is given in Table 84.

TABLE 84.—PERCENTAGE OF CASES IN WHICH THE MAXIMUM INTENSITY OF PRECIPITATION OCCURRED WITHIN VARIOUS PERIODS FROM THE BEGINNING OF THE STORM

Minutes after beginning of storm	Per cent. of cases in which maximum intensity occurred within period at		
	Washington	Savannah	St. Louis
5	17	10	31
10	38	31	61
15	59	52	69
20	64	65	74
25	72	72	76
30	81	82	78
35	86	87	80
40	91	88	88
45	93	92	93
50	94	97	98
60	100	100	100

The run-off factor gradually increases for some time after the beginning of a rain until the soil has been thoroughly saturated, and until impervious surfaces have been thoroughly wetted and the depressions filled. After that time the coefficient remains substantially constant

for a given area. It therefore makes considerable difference in the amount of run-off whether the critical precipitation comes near the beginning of a storm or after rain has been falling for some time.

It is also possible, as noted above, that if an excessive rain comes at a time when there is snow or ice upon the ground, the coefficient may be greater than unity, although this condition is so unlikely of occurrence as applied to sewer design that it may ordinarily be left out of consideration.

In their studies of rainfall and run-off, the Germans recognize three distinct coefficients, which together make up the run-off factor. These coefficients are: 1, Coefficient of distribution of rainfall; 2, coefficient of retention; 3, coefficient of retardation.

Coefficient of Distribution of Rainfall.—It is a well-recognized fact that heavy rains cover but a limited area, and the intensity of downpour diminishes as the distance from the center of the storm increases. Very little definite information is to be had regarding these matters, and that little has not been analyzed sufficiently to draw positive conclusions, other than that there is such a diminution in rate of rainfall. American engineers have usually been content to recognize the fact, and to allow for it by using a smaller run-off factor for larger areas.

Frühling states that according to observations in Breslau, Germany, the rate of precipitation at a distance of 3000 meters (10,000 ft.) from the center of the storm was one-half the maximum rate, and that the reduction in intensity was along a parabolic curve. From these data the formula $D = 1 - 0.005\sqrt{L}$ (for L in meters) has been derived, assuming the center of the storm to be in the center of the drainage area. D represents the ratio of the intensity of precipitation at a distance L from the center of the storm to that at the center. If L is expressed in feet, this reduces to $D = 1 - 0.0028\sqrt{L}$. Upon this basis the intensity becomes 0 at a distance of 7-1/2 miles from the center, or a storm may be expected to cover an area 15 miles in diameter.

Coefficient of Retention.—This coefficient takes account of the water required to wet the surfaces; evaporation during a storm; water held back in depressions and irregularities of the surface, and by vegetation, etc., and water absorbed by porous earth, which therefore does not find its way into the sewers. All of these influences have vastly more effect at the beginning of a storm than after rain has been falling for some time, and also vary with climatic conditions, so that the value of this coefficient is far from constant, even for a single drainage area. Furthermore, in growing cities the extent of the areas covered by roofs and impervious pavements is continually increasing, with a corresponding diminution of more or less pervious areas, and pavements and roofs are being made smoother and less absorbent. For this reason, present values of the coefficient are of value only for comparative purposes. It

is usually necessary to choose higher values in design, to allow for growth of the city.

According to Frühling, the values of this coefficient (assuming the surface already wetted by a previous rain) are about:

For metal, glazed tile and slate roofs	0.95
For ordinary tile and roofing papers	0.90
For asphalt and other smooth and dense pavements	0.85–0.90
For closely-jointed wood or stone block pavements	0.80–0.85
For block pavements with wide joints	0.50–0.70
For cobble stone pavements	0.4 –0.5
For macadam roadways	0.25–0.45
For gravel roadways	0.15–0.30

and for large areas, there may be assumed:

For the densely built center of the city	0.7 –0.9
For densely built residence districts	0.5 –0.7
For residence districts, not densely built	0.25–0.5
For parks and open spaces	0.1 -0.3
For lawns, gardens, meadows and cultivated areas, varying with slope and character of soil	0.05–0.25
For wooded areas	0.01–0.20

Coefficient of Retardation.—If the duration of the storm causing flood conditions is less than the time required for water to flow from the most distant point on the drainage area to the point for which computations or gagings are made, then the maximum discharge will come when less than the whole drainage area is contributing water. The ratio of the area so contributing to the total drainage area is called the coefficient of retardation.

Obviously, if the precipitation continues at a uniform rate for an indefinite time, the greatest discharge will occur when all parts of the drainage area are contributing water, and at an interval after beginning of the downpour equal to the time required for water to flow from the most distant point (measured in time of flow) to the point under consideration. If the downpour lasts but a short time, and particularly if the drainage area is irregular in shape, it is possible that the maximum discharge may occur when but a portion of the area is contributing water. This portion will be the largest area within the drainage area and between two "contours" (lines of equal "time-distance" or equal time of flow from the point under consideration) whose distance apart, measured in time, is equal to the duration of the downpour. If this time should equal the time of concentration for the entire area, the ratio would be unity and there would be no retardation.

In problems of design, it is believed that (except perhaps in the case of very large drainage areas) the maximum discharge would result from

a rain lasting for a sufficient period so that the entire area would contribute water—in other words, for a period equal to the time of concentration—and accordingly retardation should not be considered in such work. The Germans, however, seem to work from a different viewpoint, determining first the maximum duration of heavy rain that is likely to occur, and then the retardation coefficient, as well as the rate of rainfall, corresponding to this period for the drainage area under consideration.

While American engineers have, very properly, neglected retardation in design, it should not be lost sight of in studying gagings of flow in sewers. In other words, unless it is certain that the downpour has lasted for a period equal to or exceeding the time of concentration, it must be remembered that all parts of the drainage area may not have been contributing water to the maximum discharge, and the area which was actually contributing must be determined in order to find the true run-off factor.

In making this allowance for retardation, the effect of the travel of the storm should not be lost sight of. Information on this point is usually not to be had, but would be required for a complete and accurate solution of the problem.

It must not be forgotten that the time of concentration for a given drainage area is not a constant, and will be greater in light storms when the sewers are but partly filled than in heavy storms when maximum velocities are attained.

This condition, like the German "coefficient of retardation," is of importance only in studying gagings and comparing them with the storms producing the run-off, since in sewer design allowance must be made for maximum conditions.

Effect of Storage in Sewers and upon Streets and other Surfaces.—Still another element of retardation is found in the necessity of filling the sewers, gutters and other channels to a sufficient depth, and also to accumulate sufficient head to carry away the water finding its way to the drains. Thus it will be seen that a certain portion of the precipitation which is really running off is temporarily stored or retarded, and the rate of flow in the sewers is somewhat less than it would be if all the water could be conveyed away as rapidly as it is received.

Grunsky has discussed this problem at length in *Trans. Am. Soc. C. E.*, vol. lxv, p. 294, and, making certain assumptions, has elaborated the rational method of designing storm-water drains with allowance for the storage capacity of the conduits themselves. As a rule it is better in designing sewers to take no account of this storage capacity, leaving it as an additional factor of safety. The effect of such storage must, however, be borne in mind when studying gagings of storm-water flow and comparing them with precipitation records.

Values Ordinarily Assumed for Run-off Factor.—In computing from observations of rainfall and run-off the degree of imperviousness of the various kinds of surface that are found in a given urban territory, much care must be exercised in the selection of the data. Frequently the rainfall is neither uniform in intensity nor uniformly distributed over the district; in some cases the discharge is estimated from inadequate data, and in others the areas of the several classes of surface were not determined with much accuracy. As a rule, also, the time of concentration corresponding to the conditions existing at the time of gaging has not been determined, so the intensity of precipitation with which the run-off should be compared is not known. It is therefore not surprising to find wide differences in the results obtained by different observers with respect to the degree of imperviousness of the several classes of surface. The range of such variation reported in recent text-books and papers on the subject is exhibited in Table 85.

TABLE 85.—RANGE IN ESTIMATES OF RUN-OFF FROM DIFFERENT CLASSES OF SURFACE IN PROPORTION TO THE RAINFALL INTENSITY

From Bryant and Kuichling's Report on the Adequacy of the Present Sewerage System of the Back Bay District of Boston, etc., 1909

For water-tight roof surfaces	0.70 to 0.95
For asphalt pavements in good order	0.85 to 0.90
For stone, brick and wooden block pavements with tightly cemented joints	0.75 to 0.85
For same with open or uncemented joints	0.50 to 0.70
For inferior block pavements with open joints	0.40 to 0.50
For macadamized roadways	0.25 to 0.60
For gravel roadways and walks	0.15 to 0.30
For unpaved surfaces, railroad yards and vacant lots	0.10 to 0.30
For parks, gardens, lawns and meadows, depending on surface slope and character of subsoil	0.05 to 0.25

Other authorities do not attempt to make close estimates of the different kinds of surface in an urban district, but content themselves with average values of the proportional run-off, as follows:

For the most densely built-up portion of the district	0.70 to 0.90
For the adjoining well built-up portions	0.50 to 0.70
For the residential portions with detached houses	0.25 to 0.50
For the suburban portions, with few buildings	0.10 to 0.25

As mentioned in Chapter VII, Gregory has assumed that C should be considered as a variable dependent upon t, and suggests the expression: $C = 0.175t^{1/3}$ for totally impervious surfaces. The corresponding values of C for totally impervious surfaces for various times, t, in minutes would then be

t	3	5	10	15	20	30	45	60	90	120	180	186
C	0.25	0.30	0.38	0.43	0.48	0.55	0.62	0.68	0.79	0.86	0.99	1.00

Prof. H. N. Ogden gives a diagram, which is reproduced in Fig. 108, based on the results of Kuichling's studies for the City of Rochester. This diagram is based on the assumption that the percentage of impervious area in a given district bears a direct ratio to the density of population, and that for similar general conditions districts having the same density of population will have the same percentage of impervious area and the same run-off coefficient. Kuichling made a careful computation of the amount of different characters of surface in the various sewerage districts investigated by him, which Ogden has reduced to the

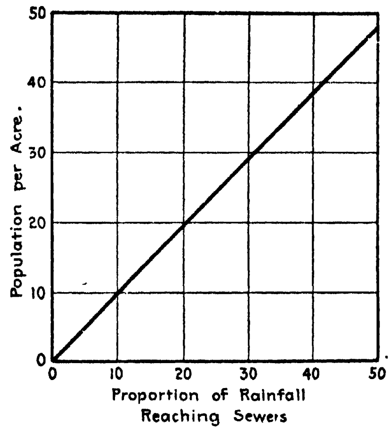


FIG. 108.—Run-off diagram based on Kuichling's studies. (Ogden.)

form shown in Table 86 which contains the information from which Fig. 108 has been prepared.

TABLE 86.—RELATION OF DENSITY OF POPULATION TO AMOUNT OF IMPERVIOUS AREA. (Ogden)

Average number of persons per acre	Percentage of fully impervious surface			Total percentage of fully impervious surface per acre
	Roofs	Improved streets	Unimproved streets and yards	
15	8.4	3.3	3.0	14.7
25	14.0	7.0	4.3	25.3
32	18.0	10.2	5.0	33.2
40	22.5	14 7	5.4	42.6
50	28 0	19 0	5 6	52.6

It should be borne in mind that these results are based upon detailed studies of only a single city, with a comparison of two or three other

places of moderate size. The figures should therefore not be considered as of general application. It has been found, however, that the percentage of area occupied by street surface in American cities bears a somewhat definite relation to the density of population, as is shown by the following diagram, Fig. 109, prepared by the authors. The information was obtained from the U. S. Census Bureau's "Statistics of Cities of over 30,000 Population for the Year 1905" and relates to 120 cities, supplemented by information from the Boston "City Record" of 1907 for the districts of that city.

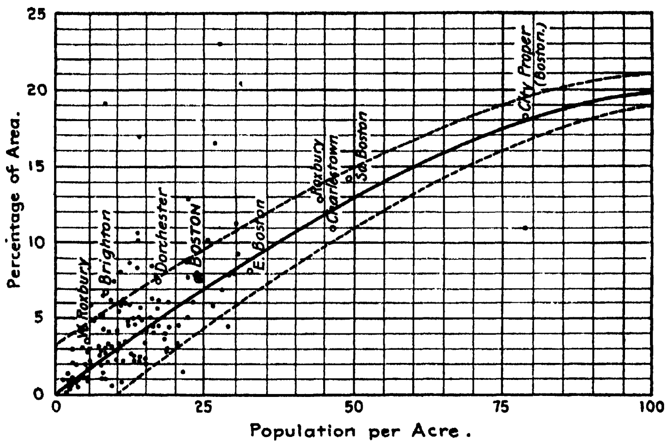


FIG. 109.—Relation between density of population and percentage of area occupied by street surfaces, not including sidewalks.

Dr. Karl Imhoff gives for ordinary German conditions the "general assumptions relating to quantity of sewage" reproduced in Table 87.

TABLE 87.—QUANTITY OF SEWAGE AND RUN-OFF IN GERMAN CITIES

Class	Conditions	Population, per acre	Quantity of house sewage,[1] c.f.s. per acre	Storm water[2]	
				Coefficient of run-off	Rate of run-off, c.f.s. per acre
I	Very thickly built up..	141	0.0116	0.80	1.37
II	Closely built up......	101	0.0083	0.60	1.03
III	Well built up.........	61	0.0050	0.25	0.43
IV	Suburban......	40	0.0033	0.15	0.26
V	Unsettled	0	0.0	0 05	0.086

[1] Based upon 100 liters per head per day, flowing off in 12 hours.

[2] Taken as 120 liters per second per hectare, equivalent to a precipitation of 1.69 inches per hour.

In general, in the absence of suitable information from which to estimate directly the run-off factor for a given area under conditions assumed to exist at the end of the "economic period of design," this factor may be most satisfactorily approximated by estimating the "equivalent percentage of totally impervious area," as it is sometimes called.

Thus, if it is assumed that in the future 15 per cent. of the district area will be covered by roofs for which the coefficient would be 0.95; 30 per cent. by pavements, with coefficient 0.90; 40 per cent. by lawns, with coefficient 0.15; 15 per cent. by gardens, with coefficient 0.10; the resulting coefficient for the entire district will be 0.4875, or, in round numbers, 0.50.

APPLICATION OF THE RATIONAL METHOD TO DESIGN

Having decided upon the time to be allowed for concentration of the water at the first inlet, and upon the coefficient to be used, the intensity of rainfall is taken from the curve adopted for this locality (discussed in detail in Chapter VI) and, since this corresponds almost exactly to the amount of the precipitation in cubic feet per second per acre,[1] the product CiA gives the amount of water to be provided for at the upper end of the sewer (first section). Having this quantity and the available grade, the sewer diameter and velocity can be determined. Dividing the distance to the next inlet by the velocity gives the increment of time, t, to be added. The area above the second point of examination and corresponding to the new t is greater than the first; the intensity of rainfall i will be less for the greater period of concentration, and the value of C may be modified; but when these elements have been determined, or assumed, the new value of Q can be obtained, and from it the required size of section.

By the use of diagrams this method can be applied rapidly and without difficulty.

Example of the Use of the Rational Method.—One of the best examples of the intelligent application of the rational method of sewer design reported in engineering literature is found in the practice at St. Louis. The following description of the application of the method in that city is adapted and amplified from an article by W. W. Horner in *Engineering News* of Sept. 29, 1910.

"An average half city block in the newer subdivisions (a residence district) is assumed. This one-half block from the center of the street to the center of the alley will be 172-1/2 ft., and from center to center of cross streets, 860 ft., giving a total area of 3.38 acres. The impervious portion of this block will be approximately:

[1] One inch per hour = 1.008 cu. ft. per second per acre.

	Sq. ft.	Per cent. of total area
Streets	20,000	13.7
Alleys	6,500	4.5
Sidewalks	6,500	4.5
House roofs	27,500	18.9
Shed roofs	4,000	2.7
Yard walks	2,000	1.4
Total	66,500	45.7

"This makes a total of 1.53 acres. The percentage of the total area which is entirely impervious will then be about 45 (the population for such a block runs about 40 per acre).

"For rain of 10 minutes duration, it was assumed that 60 per cent. of the water falling on impervious surfaces, and 20 per cent. of the water falling on lawns and hard ground, would run off. This gives an average of 38 per cent. run-off for the whole block.

"The following table (Table 88) gives the assumed percentages for each class, and the final average value of C in round numbers:"

TABLE 88.—ASSUMED PERCENTAGES OF RUN-OFF FOR ILLUSTRATION OF RATIONAL METHOD

Duration t in minutes	Per cent. run-off from		Coefficient C
	Impervious portion	Pervious portion	
10	60	20	0.40
15	70	30	0.50
20	80	35	0.55
30	85	40	0.60
60	95	50	0.70
120	95	60	0 75

These percentages of run-off are based upon the assumption that the critical rainfall or downpour of the assumed duration occurs at the beginning of a storm, before the surface has been thoroughly wetted. This is not always true, particularly for the shorter periods, and a somewhat safer basis, in many cases at least, would be to make no reduction in the coefficient for the shorter times of concentration.

Having decided upon the run-off coefficient or series of coefficients appropriate to the location under consideration, and the proper rainfall curve to be used as a basis of design (Horner's rainfall curve for St. Louis has already been shown in Fig. 93), it is convenient to construct a curve showing the product Ci for any period of concentration. This curve gives directly the discharge in cubic feet per second per acre for the duration of downpour, but applies only to the average residence

district for which the percentages were derived. For this case, the curve would be as shown in Fig. 110.

It will be noted that, for this case, with the coefficients selected as above, a uniform rate of run-off equivalent to 2.20 cu. ft. per second per acre is used for all times of concentration up to 15 minutes, and decreasing rates for longer times. Of course the selection of a rainfall curve and of suitable coefficients can only be made by one who is familiar with the locality and its records of precipitation.

Assuming that all streets and alleys are open and that no rights of way are required, the actual computations can be taken up. The first requisite is an accurate plat similar to Fig. 111; on this should be entered the elevations of the proposed or established street and alley

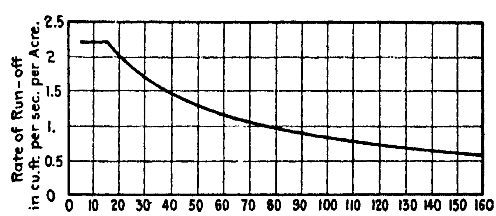


FIG. 110.—Rate of runoff for St. Louis residence districts.

grades and, if no contour map has been made, the existing surface elevations should also be shown. The street and alley inlets are then located on the plat, placed on the upper side at all street intersections and at all low points between streets, provided that the interval so established does not exceed 600 or 700 ft. When the inlets have been "spotted," the next step is to lay out the sewers to reach them, and at the same time to sewer all the private lots in the district. Obviously, the most economical layout is one which follows the natural surface slopes in the shortest line toward the outlet of the district, but also concentrates the flow as rapidly as possible. This can usually be done in more than one way, and it is often necessary to make partial designs and comparative estimates to determine which is the cheaper.

With all inlets and sewers located and the present surface elevations on all streets and alleys known, it is a comparatively easy matter to calculate the area tributary to the sewer at any point under existing conditions. This, however, is rarely of great importance in determining the maximum run-off, as the district is usually only partly built up and paved, and the percentage of run-off will be small; also much of the surface water will follow old water courses without reaching the sewers.

It is necessary for the designer to stop at this point and imagine the district as it will be when completely settled and paved. Observation of the nearest settled districts and a knowledge of the probable trend of real estate activity will enable him to estimate what class of property it will be when improved; that is, what class of houses will be built and what amount of grading may be expected in shaping up the lots. A study of the original surface and of the probable class of improvements will permit the construction of a set of minor ridge lines which will divide the small areas draining into streets from those draining into the alleys; and by these the final areas tributary to each inlet can be shown on the plat and the acreage computed. Fig. 111 shows such a map, and in Table 89 are given the drainage district numbers and the area of each district, the sum of the areas being checked against the total computed as a whole. These areas are based on an assumed condition of final grading.

TABLE 89.—DRAINAGE AREAS CORRESPONDING TO SEWER MAP SHOWN BY FIG. 111.

Area No.	Area in acres	Area No.	Area in acres
56	2.52	97	1.13
57	1.86	98	1.59
58	1.13	99	2.39
59	2.76	107	0.33
60	1.73	108	4.48
61	0.88	108-*A*	1.45
62	1.78	109	11.15
63	2.12	110	1.91
87	1.47	180	0.87
89	2.11	182	0.24
89-*A*	0.32	183	1.12
90	0.20	184	3.35
91	1.63	185	0.55
91-*A*	1.10	186	0.62
92	2.66	187	0.75
96	1.63	188	1.38
	Total = 59.21 acres.		

The preliminary sewer grades should first be drawn in at the proper depth, beginning at the lower end, as the elevation at the outlet is approximately fixed by outside conditions. Then, beginning at the upper end, the final grades can be established at the same time that the sizes are determined.

It is supposed now that the location of all inlets and the alignment and approximate grade of the sewer have been decided upon, and that the areas tributary to the sewer have been computed. There remain to be determined the amount of run-off and the size of sewers.

TABLE 90.—TABULATION OF COMPUTATIONS OF SEWER DESIGN

No.	Line from M.H.	Line to M.H.	Drainage area (district)	Actual area (A) acres	Equiv.[2] area (A') acres	S, per cent.	Length, ft.	Intensity of rain (i)	Coef. C	Ci	$Q = CiA'$ (c.f.s.)	Diam of sewer (in.)	Velocity, ft. per sec.	Total elapsed time at entrance to section (min.)	Time of flow in section (min.)	Total time to end of section[3] (min.)
1	177	176	No. 61	0 88	0.88	0.50	125	6.80	0.32	2 20	1.94	12	3.1	7 0[1]	0 7	7 7
2	176	175	No. 61	0.88	1.64	0.50	415	6.52	0.34	2.20	3.61	15	3.5	7.7	2 0	9.7
3	175	165	No. 61 + No. 187	1.63	3.05	1 00	410	5.76	0.38	2.20	6.71	15	5.1	9 7	1.3	11.0
4	166	165	No. 63	2.12	2.12	1.50	125	6.40	0.34	2.20	4.67	12	5.3	8.0[1]	0.4	8.4
5	165	164	Abv. M.H. 165	3.75	4.26	1.50	130	5.35	0.41	2.20	9.39	18	7.2	11.0	0.3	11.3
6	164	163	Abv. M.H. 165 + 60 + 62	7.26	7.26	3.00	195	5.28	0.42	2.20	15.97	18	10 1	11.3	0.3	11.6
7	172	173	No. 58	1.13	1.13	0.50	125	7.30	0.30	2.20	2.49	12	3 1	6 0[1]	0.7	6.7
8	173	174	No. 58	1.13	1.79	0.50	425	6.95	0.32	2 20	3.94	15	3 5	6.7	2 0	8.7
9	174	163	No. 58 + No. 186	1.75	3.13	3.00	400	6.12	0.36	2.20	6.89	12	7.6	8.7	0.9	9.6
10	163	162	Abv. M.H. 174 and. Abv. M.H. 164 + 59 + 107	12.10	12.10	1.00	195	5.20	0.42	2.20	26.64	24	7.3	11.6	0.4	12.0
11	162	161	Abv. M.H. 162 + dist. 56 + dist. 182	14.86	14.86	1.00	130	5.10	0.43	2.20	32.70	27	8.0	12.0	0.3	12.3
12	171	170	No. 57	1.86	1.86	1.50	120	6.40	0.34	2.20	4.09	12	5.3	8.0[1]	0.4	8.4
13	170	169	No. 57	1.86	2 61	0.75	510	6.24	0.35	2.20	5.75	15	4 3	8.4	2 0	10 4
14	169	161	No. 57	1 86	3.36	2 00	310	5 53	0.40	2.20	7.40	15	7.3	10 4	0.7	11.1
15	161	160	Total abv. M.H. 161	16.72	18.72	3.25	435	5.03	0.44	2 20	41 20	24	13.0	12.3	0.6	12.9
16	160	159	Ditto	16.72	19.83	2.50	415	4.88	0.45	2.20	43 60	27	12.3	12.9	0.6	13.5
17	131	194	....	0	0.82	2 00	310	8.00	0.27	2.20	1.88	12	6.1	5.0[1]	0.8	5.8
18	194	197	No. 183	1.12	2.77	0.60	500	7.44	0.30	2.20	6.10	18	4.4	5.8	1.9	7.7
19	197	257	No. 183	1.12	2.77	2.00	160	6.52	0.34	2.20	6.10	12	7.3	7.7	0.4	8.1
20	257	159	No. 183 + 184 + 185	5.02	5.55	5.00	165	6.36	0.35	2.20	12.20	15	11.8	8.1	0.2	8.3
21	159	(a)	108 + area abv. M.H. 159	26.22	28.07	2.00	185	4.73	0.47	2.20	61.80	26×39	12.4	13.5	0.3	13.8

TABLE 90.—TABULATION OF COMPUTATIONS OF SEWER DESIGN (*Continued*)

No.	Line from M.H.	Line to M.H.	Drainage area (district)	Actual area (A) acres	Equiv.[2] area (A') acres	S, per cent.	Length, ft.	Intensity of rain (i)	Coef C	Ci	$Q = CiA'$ (c.f.s.)	Diam. of sewer (in.)	Velocity, ft. per sec.	Total elapsed time at entrance to section (min.)	Time of flow in section (min.)	Total time to end of section[3] (min.)
22	168	167	108A + 89*A*	1.77	2.36	4.75	440	6.80	0.32	2.20	5.20	12	9.6	7.0[1]	0.8	7.8
23	167	(*a*)	89 + 89*A* 108 + 108*A*	8.36	8.36	29.30	40	6.48	0.34	2.20	18.40	12	......	7.8	0	7.8
24	(*a*)	157	Abv. (*a*) + 90	30.30	32 73	2.56	400	4.65	0.47	2.20	72.00	26×39	13 5	13.8	0.5	14.3
25	157	(*b*)	Ditto	30.30	33.33	1.50	225	4 54	0.49	2.20	73.30	30×45	11.8	14.3	0.3	14.6
26	(*b*)	(*c*)	Abv. 152 + 92 + 109	44.11	45.48	1.00	220	4.48	0.49	2 20	100.00	34×51	10 6	14.6	0.3	14.9
27	274	275	....	0	0.54	1.50	405	8.00	0.27	2.20	1.19	12	5 3	5.0[1]	1.3	6.3
28	275	270	See note	3 00	4.08	0.80	435	7.15	0.31	2.20	8.98	18	5 2	6.3	1.4	7.7
29	365	270	180	0.87	0.87	2 00	165	8.00	0.27	2.20	1.95	12	6.2	5.0[1]	0.4	5.4
30	276	270	....	0	1.12	1.25	420	8.00	0.27	2.20	2.46	12	4.9	5.0[1]	1.4	6.4
31	270	269	180 + 109	12.02	13.14	6.00	125	6.52	0.34	2.20	28.90	18	14.5'	7.7	0.1	7.8
32	269	(*c*)	As at M.H. 270 + 110	13.93	14.36	16.70	40	6.48	0 34	2.20	31.60	18	......	7.8	0	7.8
33	(*c*)	155	See notes	46.89	48.26	1.00	125	4.42	0.49	2 18	105.00	36×54	10 8	14.9	0.2	15.1
34	271	272	....	0	0.53	2.50	400	8.00	0.27	2.20	1.19	12	7.0	5.0[1]	1.0	6.0
35	272	155	188	1.38	1.71	4.75	440	7.30	0.30	2.20	3.76	12	9.5	6.0	0.8	6.8
36	273	277	....	0	0.82	1.50	310	8.00	0.27	2.20	1.81	12	5.3	5.0[1]	1.0	6.0
37	277	155	99	2.39	2.66	1.00	150	7.30	0.30	2.20	5.85	15	5.2	6.0	0.5	6.5
38	155	(*d*)	Abv. (*c*) + 91 + 108 + 99	52.29	54.36	1.00	150	4.38	0.49	2.16	117.00	36×54	10.8	15.1	0.2	15.3
39	(*d*)	154	+ 87 + 91*A*	54.86	57.10	1.00	175	4.35	0.49	2.14	122.00	36×54	10 8	15.3	0 3	15.6
40	154	(*e*)	Same	54.86	57 29	1.00	140	4.31	0.50	2.13	122.00	36×54	10 8	15.6	0.2	15.8
41	(*e*)	153	+ 97 + 98	57.58	59.71	1.00	300	4.28	0 50	2.12	127.00	36×54	10 8	15.8	0.5	16.3
42	153	136	+ 96	59.21	61.34	1.00	160	4.22	0.50	2.11	129.00	36×54	10.8	16.3	0.3	16.6
		At main line														

[1] Assumed. [2] Including allowance for roofs drained directly. [3] The numbers in this column are the sums of those in the two columns immediately preceding it.

Table 90 contains the figures for the design of sewers in the district shown in Fig. 111. In this table, the sewer lines are denoted by the numbers of the manholes at the ends.

The detailed computation of this example is as follows:

1. District No. 61 drains to inlet near M. H. 177; area 0.88 acre. Time required for concentration at inlet assumed at 7 minutes. Ci taken from curve as 2.20. Then $Q = CiA' = 1.94$ c.f.s. On Fig. 112[1] start with $Q = 1.94$, follow vertically to curve of $S = 0.005$ (slope fixed in advance), finding size just under 12 in. On 12-in. line follow horizontally to the right to velocity curve for $S = 0.005$, finding $v = 3.1$. For length 125 ft. and $v = 3.1$, time of flow = 40 seconds = 0.7 minutes. Then t at M. H. 176 = 7.7 minutes.

2. The section between M. H. 176 and 175 contains no inlets, hence there is no increase of surface area draining, which remains 0.88 acre. Roof connections from houses in district No. 61 fronting on Emerson Ave. have been allowed for in assuming the coefficient C; but roof water will also be received from houses facing on Allcott and Emerson Aves., each side of district No. 187, and from houses on Allcott Ave. above district No. 61. As drainage of district 187 is not yet accounted for, it is assumed that roof water from all lots opposite district 187 is taken in, a distance of 500 ft. for both sides of the street. Since half of the lot area on Allcott Ave. at the westerly end has already been included in district No. 61, one-half of the length of 140 ft. is assumed to be contributing roof water. Then there is roof water from $500 + (1/2 \times 140) = 570$ ft., which, at 0.00133 acre[2] per foot, corresponds to 0.76 acre. The total equivalent area (A') for design is then $0.88 + 0.76 = 1.64$ acres. Then $Q = 3.61$, diameter = 15 in., $v = 3.5$, and time of flow = 2.0 minutes, making total time to M. H. 175 = 9.7 minutes.

3. Section between manholes 175 and 165. District No. 187 drains into sewer at M. H. 175, making total surface area draining above M. H. 165, 1.63 acres. Roof water will be received from both sides of this section of sewer below district No. 187 and above M. H. 165—making $2 \times 360 = 720$ ft. This section lies in district No. 63, the surface water from which has not yet entered the sewer, and will also receive roof water from part of district No. 62. Roof water from half the area fronting on Allcott Ave. above district No. 61 should be included, as in (2), because but half the area of these lots has yet been accounted for; this calls for $1/2 \times 140 = 70$ ft. Roof water from half the lots each side of district 187 must also be included; this calls for $1/2 \times 550$[3] $= 275$ ft. (But one-half this area is included, as the

[1] Horner's St. Louis diagram for determining sewer sizes is used in this example, and is reproduced on page 290.

[2] In arriving at this figure it is assumed that 80 per cent. of the block frontage is or will be occupied by houses, and that they are 45 ft. deep, so that for each linear foot of street we shall have on each side a roof area of $0.80 \times 45 = 36$ sq. ft. But the coefficient of run-off from roof surfaces will be at least 0.80, while that from the average surface area, including lawns, street surfaces, roofs, etc., has been assumed at 0.50 for a 15-minute downpour. Then the run-off from 36 sq. ft. of roof area corresponds to that from $36(80/50) = 58$ sq. ft. approximately = 0.00133 acre of average block surface.

[3] Note that the boundary of the drainage district is 25 ft. beyond M. H. 175.

other half has already been accounted for by including district 187 in the drainage area A.) Roof water is then computed from $720 + 70 + 275 = 1065$ ft.; $1065 \times 0.00133 = 1.42$. $1.42 + 1.63 = 3.05 = A'$.

The other values are then found as shown in the tabulation.

4. M. H. 166 to 165. Drainage district No. 63, through inlet near M. H. 166. Although some of the roofs in this area have already been included as draining into sewer between M. H. 175 and 165, it must be remembered that we are designing for future conditions. Very likely many of the roofs allowed for are not yet built. So no deduction is made from area No. 63, even though we have already allowed in the computations that some of the rain falling upon it drains elsewhere. Then $A = A' = 2.12$, and for $S = 1.50$ we have from diagram, $d = 12$, $v = 5.3$, $t = 0.4$ minutes.

5. M. H. 165 to 164. Area above 165 = districts 61, 63 and 187. Area $= 1.63 + 2.12 = 3.75 = A$.

In view of the fact that roofs on Allcott Ave. are expected to drain to sewer between M. H. 176 and 165, while half of the area of the corresponding lots is already included in districts 61, 187, and 63, assume that half the roof water in district 62 (length 770 ft.) reaches the sewer above M. H. 165. (This assumption has already been made in computations 2 and 3.)

$770 \times 1/2 \times 0.00133 = 0.51$ acre. $A' = 3.75 + 0.51 = 4.26$. Then $Q = 9.39$ and from diagram, if $S = 1.50$, $d = 18$ in., $v = 7.2$, and $t = 0.3$ minute.

6. M. H. 164 to 163. Area A = that in computation 5 plus districts 62 and 60 which drain into catchbasins near M. H. 164. Therefore $A = 3.75 + 1.73 + 1.78 = 7.26$. No further allowance for roof water is to be made, so $A' = A$. Then $Q = 15.97$, and for $S = 3.00$, $d = 18$, $v = 10.1$, $t = 0.3$.

7. M. H. 172 to 173. Area A = district 58 = 1.13 acres. Nothing additional for roof water. Therefore $A' = A$. $S = 0.50$; then from diagram, we obtain the figures tabulated.

8. M. H. 173 to 174. Area A = district No. 58 only = 1.13. For roofs, allow

1/2 of 85 ft. for section of Allcott Ave. opposite district 58	= 42.5 ft.
1/2 of 275 ft. for section of Allcott Ave. opposite district 186	= 137.5 ft.
1/2 of 75 ft. for section of Davison Ave. opposite district 186 and east of district 58	= 37.5 ft.
All of 275 ft. for roof water from district 186	= 275.0 ft.
	492.5 ft.

492.5 at 0.00133 acre = 0.66 acre. Then $A' = 1.13 + 0.66 = 1.79$. Then $Q = 2.20 \times 1.79 = 3.94$, and for $S = 0.50$, $d = 15$ in., $v = 3.5$, $t = 2.0$ minutes.

9. M. H. 174 to 163. A = that of computation 8 + district 186 draining into M. H. 174, making $1.13 + 0.62 = 1.75$ acres. Roof water must be allowed for

$$\left.\begin{array}{r}\text{410 ft. on Allcott Ave. below district 186} \\ \text{410 ft. on Davison Ave. below district 58}\end{array}\right\} = 820 \text{ ft.}$$

1/2 of 75 ft. on Davison Ave. below district 58 = 37 ft.
1/2 of 360 ft. on Allcott Ave. opposite districts 58 and 186 = 180 ft.

1037 ft.

$0.00133 \times 1037 = 1.38$. $A' = 1.75 + 1.38 = 3.13$; then $Q = 6.89$ and for $S = 3.0$, $d = 12$ in., $v = 7.6$, $t = 53$ seconds = 0.9 minute.

10. M. H. 163 to 162. A = district 59 + district 107 + areas above M. H. 174 and 164. $A = 2.76 + 0.33 + 1.75$ (comp. 9) $+ 7.26$ (comp. 6) $= 12.10$ acres. No additional roof areas. Then $Q = 2.20 \times 12.10 = 26.64$, and for $S = 1.0$, from diagram, $d = 24$ in., $v = 7.3$, $t = 27$ seconds = 0.4 minute.

11. M. H. 162 to 161. A = same as in comp. 10 + district 56 + district 182, both these districts draining to catchbasins near M. H. 162. Then $A = 12.10 + 2.52 + 0.24 = 14.86$. No additional roof water areas. Therefore, $A' = A$. Then $Q = 2.20 \times 14.86 = 32.70$. Then if $S = 1.00$, $d = 2.7$ in., $v = 8.0$, $t = 16$ seconds = 0.3 minute.

12. M. H. 171 to 170. A = district 57, to catchbasins near M. H. 171. $A = 1.86$. No additional roof area. Therefore, $A' = A$. Then $Q = 2.20 \times 1.86 = 4.09$ and for $S = 1.5$, $d = 12$ in., $v = 5.3$, $t = 23$ seconds = 0.4 minute. Assume time of concentration at catchbasin = 8 minutes.

13. M. H. 170 to 169. No additional surface area, therefore, $A = 1.86$ as before. Roofs on the half-lots on Beacon Ave., opposite district 57, and on the whole lots in districts 55 and 56 between district 57 and M. H. 169 must be allowed for, $1/2 \times 280 + 210 + 210 = 560$ ft. $560 \times 0.00133 = 0.75$ acre. $A' = A + 0.75 = 1.86 + 0.75 = 2.61$ acres. Then $Q = 2.20 \times 2.61 = 5.75$, and for $S = 0.75$, $d = 15$ in., $v = 4.3$, $t = 119$ seconds = 2.0 minutes.

14. M. H. 169 to 161. No additional surface drainage admitted, therefore, $A = 1.86$ as above. For roofs, allow same as in comp. 13 plus roofs on both sides, full lots, from M. H. 169 to street next to right or 2×280 ft. = 560 ft. $560 \times 0.00133 = 0.75$. Then $A' = 2.61$ (from comp. 13) $+ 0.75 = 3.36$. $Q = 2.20 \times 3.36 = 7.40$. For $S = 2.00$, $d = 15$ in., $v = 7.3$, $t = 42$ seconds = 0.7 minute.

15. M. H. 161 to 160. Drainage area = everything above M. H. 161 (from comp. 11) = 14.86 acres; (from comp. 14) 1.86 acres; no additional surface area; then $A = 14.86 + 1.86 = 16.72$ acres. For roofs, whole lots both sides of sewer between M. H. 161 and 160, or $2 \times 435 = 870$ ft. $0.00133 \times 870 = 1.16$ acres. $16.72 + 1.16 = 17.88$ acres.

In addition, there may be roof water from lots facing Beacon Ave., along district 55, as allowed for in computation 14, amounting to 0.84 acre in all. Since district 55 drains into another branch of the system, this allowance of 0.84 acre is not included in any additions of surface area drained, and must be added in all estimates of A' along the main line (receiving drainage below M. H. 161).

Then $A' = 17.88 + 0.84 = 18.72$ acres. $Q = 2.20 \times 19.72 = 41.2$ and if $S = 3.25$, $d = 24$ in., $v = 13$, $t = 33$ seconds $= 0.6$ minute.

16. M. H. 160 to 159. No additional surface area; therefore, $A = 16.72$. For roofs, include the same as in computation 15 + allowance for both sides of the section 160–159, or $2 \times 415 = 830$ ft. $0.00133 \times 830 = 1.11$ acres. Then $A' = 18.72 + 1.11 = 19.83$. $Q = 2.20 \times 19.83 = 43.6$, and for $S = 2.5$, $d = 27$ in., $v = 12.3$, $t = 34$ seconds $= 0.6$ minute.

17. M. H. 131 to 194. No surface area draining into M. H. 131, therefore, $A = 0$. Roofs must be allowed for on both sides and for the whole distance $2 \times 310 = 620$ ft. $0.00133 \times 620 = 0.82$ acre. Then $Q = 2.20 \times 0.82 = 1.88$, and if $S = 2.0$, $d = 12$ in., $v = 6.1$, $t = 51$ seconds $= 0.8$ minute. No surface water inlet, and storm water received only from roofs, which are very quick; assume 5 minutes from time rain falls until it reaches sewer.

18. M. H. 194 to 197. Surface water from district 183 admitted at M. H. 194; therefore, $A =$ district $183 = 1.12$ acres. Roof water from 380 ft. on south side and also from half of 420 ft. (outside of district 183). On north side there will be roof water from 650 ft. In all, $650 + 210 + 380 = 1240$ ft. $0.00133 \times 1240 = 1.65$ acres. $1.12 + 1.65 = 2.77$ acres $= A'$. $Q = 2.20 \times 2.77 = 6.1$ and for $S = 0.60$, $d = 18$ in., $v = 4.4$, $t = 114$ seconds $= 1.9$ minutes.

19. M. H. 197 to 257. No additional surface water inlets, therefore, $A = 1.12$ as before. No additional roof inlets, therefore, $A' = 2.77$ as before. Then $Q = 6.10$ as before, and for $S = 2.0$, $d = 12$ in., $v = 7.3$, $t = 22$ seconds $= 0.4$ minute.

20. M. H. 257 to 159. Drainage area is increased by districts 184 and 185, through inlets near M. H. 257. $1.12 + 3.35 + 0.55 = 5.02$ acres $= A$. Add for roof water from 1/2 of the lots fronting Allcott Ave. in district 172. $1/2 \times 800 \times 0.00133 = 0.53$ acre. This must be included in all succeeding designs until district 172 has been included. Then $A' = 5.02 + 0.53 = 5.55$. Then $Q = 2.20 \times 5.55 = 12.20$, and for $S = 5.00$, $d = 15$ in., $v = 11.8$, $t = 14$ seconds $= 0.2$ minute.

21. M. H. 159 to point *a*. Drainage area = everything above M. H. 159, plus district 108 (inlet near M. H. 159). 16.72 above 161, plus 4.48 = (district 108), plus 5.02 above $257 = 26.22$ acres $= A$.

For roof water, add the sections outside the direct drainage area noted above, $0.84 + 0.53 = 1.37$ acres. Also add roof water from district 108 A, $0.00133 \times 360 = 0.48$ acre making total roof allowance 1.85 acres. Thus $A' = 26.22 + 1.85 = 28.07$ acres. $Q = 2.20 \times 28.07 = 61.8$, and for $S = 2.00$, $d = 26 \times 39$ in., $v = 12.4$, $t = 15$ seconds $= 0.3$ minute.

22. M. H. 168 to 167. Areas 108A and 89A, inlets near M. H. 168, 0.32 plus $1.45 = 1.77$ acres $= A$. For roof water, inlets from one side only, $440 \times 0.00133 = 0.59$ acre. (Note that house inlets indicate drainage from only one side.) $1.77 + 0.59 = 2.36$ acres $= A'$. $Q = 2.20 \times 2.36 = 5.20$ c.f.s., for $S = 4.75$, $d = 12$ in., $v = 9.6$, $t = 46$ seconds $= 0.8$ minute. Assume time of concentration at inlets = 7 minutes.

23. M. H. 167 to point *a*. Area A = as in (22) + districts 89 and 108. (This assumes the whole of district 108 draining to inlet near M. H. 167, although the whole of it has previously been considered at inlet near 159.

Either case is possible, although usually each inlet would receive part of the water.) Then $A = 1.77 + 2.11 + 4.48 = 8.36$ acres. No additional roof water, therefore, $A' = A$. Then $Q = 2.20 \times 8.36 = 18.4$ and for $S = 29.30$, $d = 12$ in., v is beyond limits of diagram and t is negligible.

24. From point a to M. H. 157. Area includes district 90, through inlet at northeast corner of Beacon and Theokla Aves. Then $A = (8.36 + 26.22 + 0.20 = 34.78) -$ (district 108 counted twice $= 4.48$) $= 30.30$ acres. For roof water add the constant items shown above, amounting to 1.37 acres, as in computation 21; also for sides of the section of sewer under consideration $2 \times 400 \times 0.00133 = 1.06$ acres. Then $A' = 30.30 + 1.37 + 1.06 = 32.73$. Then $Q = 2.20 \times 32.73 = 72.0$, and for $S = 2.56$, $d = 26 \times 39$ in., $v = 13.5$, $t = 30$ seconds $= 0.5$ minute.

25. From M. H. 157 to point b (catchbasin inlets). No additional surface area drained, therefore, $A = 30.30$ as before. A' is same as in computation (24) + allowance for 225 both sides, between 157 and b, $0.00133 \times 2 \times 225 = 0.60$ acre. Then $A' = 32.73 + 0.60 = 33.33$. Then $Q = 2.20 \times 33.33 = 73.3$ and for $S = 1.50$, $d = 30 \times 45$ in., $v = 11.8$, $t = 19$ seconds $= 0.3$ minute.

26. From point b to point c. Drainage area = the total above M. H. 157 (which is 30.30) + districts 92 and 109, with inlets at point b. The whole of district 109 is included here, although in ordinary times a part, and sometimes the whole, would be admitted at M. H. 273 and 275. Then $A = 30.30 + 2.66 + 11.15 = 44.11$. Roof water is fully allowed for by the inclusion of districts 92 and 109, except the sections outside the drainage area and noted above, amounting to 1.37 acres. Then $A' = 44.11 + 1.37 = 45.48$. $Q = 45.48 \times 2.20 = 100$. For $S = 1.0$, $d = 34 \times 51$ in., $v = 10.6$, $t = 21$ seconds $= 0.3$ minute.

27. From M. H. 274 to 275. No surface inlets; therefore $A = 0$. Roof water from one side of the sewer, 405 ft. long, $0.00133 \times 405 = 0.54$ acre $= A'$. Then $Q = 2.20 \times 0.54 = 1.19$, and for $S = 1.5$, $d = 12$ in., $v = 5.3$, $t = 77$ seconds $= 1.3$ minutes.

28. From M. H. 275 to 270. Estimate that surface water from 3 acres of district 109 will enter at M. H. 275, in addition to allowance for roof water, which will be that in computation 27 and allowance for one side, 400 ft., between M. H. 275 and 270, 0.54 acre. Then total roof allowance $= 1.08$ acres, and $A' = 4.08$. Then $Q = 2.20 \times 4.08 = 8.98$ and for $S = 0.8$, $d = 18$ in., $v = 5.2$, $t = 84$ seconds $= 1.4$ minutes.

29. From M. H. 365 to 270. No roofs. Then $Q = 2.20 \times 0.87 = 1.95$ c.f.s. For $S = 2.0$, $d = 12$ in., $v = 6.2$, $t = 27$ seconds $= 0.4$ minute. Catchbasin is centrally located in district, therefore assume time of concentration $= 5$ minutes.

30. From M. H. 276 to 270. No surface water. For roofs, both sides of 420 ft., $0.00133 \times 840 = 1.12$ acres. $Q = 1.12 \times 2.20 = 2.46$. For $S = 1.25$, $d = 12$ in., $v = 4.9$, $t = 86$ seconds $= 1.4$ minutes. Assume time of concentration 5 minutes as it is wholly on roofs.

31. From M. H. 270 to 269. Area, district 180 and district 109 (assumed to drain through M. H. 274, 275, and 270). Then $A = 11.15 + 0.87 = 12.02$. For roofs, we have the allowances in computation 30.

Then $A' = 12.02 + 1.12 = 13.14$. Then $Q = 2.20 \times 13.14 = 28.9$ and for $S = 6.0$, $d = 18$ in., $v = 14.5$, $t = 9$ seconds $= 0.1$ minute.

32. From M. H. 269 to point c. Area = that above M. H. 270 + district 110, $A = 12.02 + 1.91 = 13.93$ acres. For roofs, only the part of the line 276 − 270 lying without district 110 has to be added. $2 \times 200 = 400$ ft. (for both sides) $400 \times 0.00133 = 0.43$ acre. Then $A' = 13.93 + 0.43 = 14.36$, $Q = 2.20 \times 14.36 = 31.6$, $d = 18$ in., $v = 20$, t = negligible.

33. From point c to M. H. 155. Area above b, 44.11, plus area above 269, 13.93 = 58.04; deduct for district 109 included twice, 11.15, leaves 46.89 = A. No roofs in this section, but the areas noted above must be included, amounting to 1.37 acres. Then $A' = 46.89 + 1.37 = 48.26$ acres. $Q = 2.18$[1], $48.26 = 105$. For $S = 1.0$, $d = 36 \times 54$ in., $v = 10.8$, $t = 12$ seconds $= 0.2$ minute.

34. From M. H. 271 to 272. No surface area. Roofs from one side of 400 ft. $400 \times 0.00133 = 0.53$ acre $= A'$. For $S = 2.5$, $v = 7.0$, $d = 12$ in., $t = 400/7 = 57$ seconds $= 1.0$ minute. Time of concentration taken as 5 minutes from roofs only.

35. From M. H. 272 to 155. The direct drainage area is district No. 188; $A = 1.38$. Roof water is to be taken (or allowed for) on one side of this section of sewer; $0.00133 \times 400 = 0.56$ acre. $A' = 1.38 + 0.53 = 1.71$. $Q = 2.20 \times 1.71 = 3.76$, and for $S = 4.75$, $d = 12$ in., $v = 9.5$ and $t = 46$ seconds $= 0.8$ minute.

36. From M. H. 273 to 277. No surface drainage. Roofs both sides of entire length $= 2 \times 310 \times 0.00133 = 0.82$ acre. Then $A' = 0.82$. $Q = 2.20 \times 0.82 = 1.81$. For $S = 1.50$, $d = 12$ in., $v = 5.3$, $t = 59$ seconds $= 1.0$ minute.

37. From M. H. 277 to 155. District 99 drains mainly to catchbasin near M. H. 155, but partly to M. H. 277; for safety, assume entire drainage at M. H. 277. Then A = district 99 = 2.39 acres. For roofs we have also the half-lots in district 98 minus a length of about 400 ft.; making $200 \times 0.00133 = 0.27$ acre. Then $A' = 2.39 + 0.27 = 2.66$. Then $Q = 2.20 \times 2.66 = 5.85$. For $S = 1.00$, $d = 15$ in., $v = 5.2$, $t = 29$ seconds $= 0.5$ minute.

38. M. H. 155 to point d. Drainage area = everything to M. H. 155 = 46.89 + districts (91 + 188 + 99), $46.89 + 1.63 + 1.38 + 2.39 = 52.29$ acres $= A$. For roofs there is allowance for the half lots in computation 37, besides the sections permanently brought forward in main line, $0.27 + 1.37 = 1.64$ acres additional; also roof areas draining to the section of sewer between M. H. 276 and 270, without district 110, amounting to 0.43 acre as in computation 32. Then $A' = 52.29 + 1.64 + 0.43 = 54.36$, $Q = 2.16 \times 54.36 = 117$ c.f.s. Then for $S = 1.0$, $d = 36 \times 54$ in., $v = 10.8$, $t = 14$ seconds $= 0.2$ minute.

39. From point d to M. H. 154. Area A is increased by districts 87 and 91A, therefore $A = 52.29 + 1.47 + 1.10 = 54.86$. Total roof allowance for parts of districts outside those discharging into this branch through surface inlets, as in computation 38, $= 1.37 + 0.43 = 1.80$ acres. Additional roof water from the half lots included in computation 38, 0.27 acre,

[1] Total elapsed time is such that Ci is less than 2.20.

and from one side of the sewer from d to 154, $130 \times 0.00133 = 0.17$ acre, is also to be included. Then $A' = 54.86 + 1.80 + 0.27 + 0.17 = 57.10$. Then $Q = 57.10 \times 2.14 = 122$, and for $S = 1.0$, $d = 36 \times 54$ in. (the excess is nearly enough to call for the next size); $v = 10.8$, $t = 16$ seconds $= 0.3$ minute.

40. From M. H. 154 to point e. No increase in area A. Increase in roof allowance for 140 ft. of sewer, one side, $140 \times 0.00133 = 0.19$ acre. Then $A' = 57.10$ (from computation 39) $+ 0.19 = 57.29$. Then $Q = 2.13 \times 57.29 = 122$ c.f.s. For $S = 1.0$, $d = 36 \times 54$ in., $v = 10.8$, $t = 13$ seconds $= 0.2$ minute.

41. From point e to M. H. 153. Drainage area increased by districts 97 and 98, and $A = 54.86 + 1.13 + 1.59 = 57.58$. For roofs add 1.80, as previously noted; also allowances for one-half of one side along 400 ft. of sewer $= 0.27$ for half lots in district 96, and for roofs in the area opposite the ends of districts 96 and 97—125 ft., 0.16 acres. Then $A' = 57.58 + 0.27 + 0.16 + 1.80 = 59.81$ acres. Then $Q = 2.12 \times 59.81 = 127$. And for $S = 1.0$, $d = 36 \times 54$ in. (nearly to next larger size), $v = 10.8$, $t = 28$ seconds $= 0.5$ minute.

42. From M. H. 153 to main sewer at M. H. 136. Area increased by district 96, therefore $A = 57.58 + 1.63 = 59.21$. For roofs add the outside areas (1.80 acres) as before, also allowance for lots east of districts 94, 96 and $97 = 250$ ft. $250 \times 0.00133 = 0.33$. Then $A' = 59.21 + 1.80 + 0.33 = 61.34$ and $Q = 2.11 \times 61.34 = 129$. For $S = 1.0$, $d = 36 \times 54$ in., $v = 10.8$, $t = 15$ seconds $= 0.3$ minute.

Another Application of the Rational Method.—Substantially the same method, with only minor differences in the manner in which it is developed, is followed in the Public Works Department of Boston. The following description of the procedure has been prepared by F. A. Lovejoy, assistant engineer, to whom, and to E. S. Dorr, engineer of the Sewer Service, acknowledgments are due for information and diagrams furnished and for permission to make free use of them.

1. Lay out line of main drain above point where size is required to extreme upper end of area, determining roughly the principal hydraulic grade points with lengths between, and location of the most important branches.

2. Select a number of points on the main drain where the velocity of flow is likely to change considerably, either from change in the hydraulic gradient, or from increase in tributary area from entrance of large branches. Place these points in their order over the columns on the Schedule (page 288) beginning with the highest.

3. Fill out items in Schedule in order from top to bottom.

"Time allowance" is the estimated time required for the storm water to reach the main drains from roofs, gutters, etc. Five to fifteen minutes is commonly used. This item is to be added to the estimated time of flow in drains to give total time used in calculating rain intensity (R_i).

Formula (R_i) = rate of rainfall in inches per hour, or cubic feet per

second per acre. For Mr. Dorr's curve, $R_i = 150/(t + 30)$ in which t equals total time in minutes before mentioned.

DISTRICT.................... DATE.................191.... NO.......... SHEET..........

AREA.................... CALCULATED BY.......... CHECKED BY.......... DISTRICT ENGINEER..........

ELEVATION, H.G. AT POINT														
TIME ALLOWANCE = MIN.														
FORMULA, R_1 =														
	ABOVE	BELOW	ABOVE	BELOW	ABOVE	BELOW	ABOVE	BELOW	ABOVE	BELOW	ABOVE	BELOW	ABOVE	BELOW
TOTAL TRIBUTARY AREA (ACRES)														
SURFACE COEFFICIENT														
SURF. X RETENTION COEFF.														
EFFECTIVE AREA (ACRES)														
LENGTH OF DRAIN (FEET)														
TOTAL LENGTH OF DRAIN (FEET)														
HYDRAULIC GRADIENT, FALL														
ASSUMED V. UPPER END														
ASSUMED V, LOWER END														
MEAN V, (FT.SEC.)														
TIME IN DRAIN MINUTES														
TOTAL TIME MINUTES														
RAINFALL RATE (R_1)														
DISCHARGE, (CU.FT.SEC.)														
SIZE REQUIRED (CIRCULAR, n=.015)														
CALCULATED V, (FT.SEC.)														
CONSTRUCTION SECTION, PIPE														
" HORSE-SHOE (STRAIGHT)														
" " (CURVED)														
CALCULATED Q (CU.FT.SEC. n=.015)														

Blank schedule used in tabulating storm-water drain data, Boston.

The total tributary area is determined just above and just below the points selected and placed at head of columns.

The effective area is the total tributary area multiplied by the percentage of run-off expected. This may be estimated by dividing the area into impervious and pervious portions, taking say 100 per cent. of the impervious area, consisting of roofs, asphalt streets, paved yards, etc., and adding a smaller percentage of the pervious area according to judgment; say 15 per cent. for grass lawns and 30 or 40 per cent. for dirt, etc. The slope of the surfaces should also be considered.

The percentage of imperviousness of the total tributary area is found by adding together the impervious portions of the various types of area estimated as above, and dividing the sum by the total area. A further correction, depending upon the time of contribution, may then be applied. This correction is based on the theory that both the percentage of absorption of the pervious surfaces, and the percentage of storage on the impervious surfaces are greatest at the beginning of a storm and that the percentage of run-off therefore increases with the duration of the storm. More observations are needed to determine this factor properly. A diagram has, however, been designed to use for this correction until such observations are made.

Length of drain is taken from the point next above to the one under consideration.

"Assumed V" at upper end, in case of the first upper section, is taken as the velocity of flow in feet per second in a 10 or 12-in. pipe at the determined hydraulic grade. In other cases it is the same as the velocity from the outlet of the previous section.

"V" at lower end is first roughly taken from a sewer diagram based on Kutter's formula, Fig. 113, by setting scale on proper hydraulic grade and guessing at probable discharge. The calculation is then finished and with the discharge found, a new V at outlet is more carefully calculated. This process is to be repeated if necessary until the proper V is found.

"Mean V" is the average of the V at upper and lower ends.

$$\text{Time in drain} = \frac{\text{length of drain}}{\text{mean } V \times 60} = \text{minutes.}$$

Total time = sum of times in mains to point taken + time allowance.

Rainfall rate determined by formula.

Discharge = cubic feet per second = $R_t \times$ effective area.

For approximate estimates by this method, for areas not exceeding 10 acres, Lovejoy has devised the very ingenious diagram illustrated in Fig. 113. As shown in the illustration, an effective area of 6 acres might yield a storm flow of 14-1/4 cu. ft. per second in a district where the mean slope is 1 in 600, and a 28-in. sewer flowing full would be required to care for this run-off.

COMPARISON OF DESIGNS BY THE RATIONAL METHOD AND BY McMATH FORMULA

Since McMath's formula was derived from observations made in St. Louis, and especially for use in that city, it becomes particularly interesting to compare the results obtained by the application of

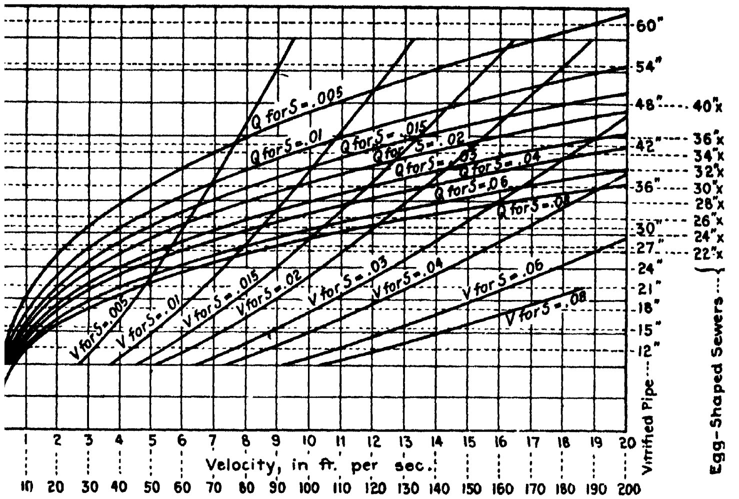


FIG. 112.—Approximate curves for sewer design, St. Louis.

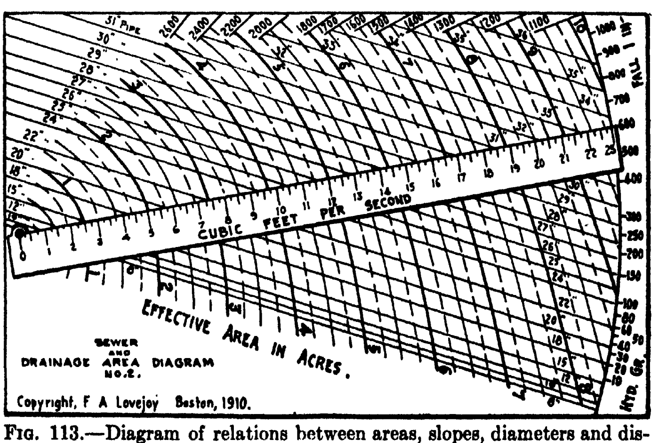


FIG. 113.—Diagram of relations between areas, slopes, diameters and discharges of sewers.

Reproduced by permission of F. A. Lovejoy.

Directions:—Set scale on hydraulic gradient; where straight lines cut scale, read capacity of pipe sewer running full. For storm run-off, set scale on mean hydraulic gradient of system; curved lines give maximum flow for given effective area. For effective area use only the impervious portion of tributary area, *i.e.*, 100 per cent. of roofs, asphalt, etc.; 80 per cent. of first-class roadways, sidewalks, etc.; 15 to 40 per cent. of hard dirt or clay; omit flat grass land and porous soils. For very long, narrow areas take 10 per cent. from reading; for very short ones add 10 per cent.

McMath's formula with those obtained by the rational method of design for St. Louis. Horner says (*Engineering and Contracting*, Sept. 13, 1911):

"Some of the sewers designed by the McMath formula have since been found to be overcharged, but as few computations have been preserved it is difficult to tell how the formula was applied and how far the personal equation entered into the design."

It is, however, significant to compare Horner's own designs by the rational method, as given in the example quoted, and summarized in Table 90, with designs for the same district by the McMath formula, using the values recommended by McMath for St. Louis, namely $e = 0.75, i = 2.75, S = 15$.

It will be seen from Table 91 that, using Horner's data for the area of drainage districts and for additional allowance for roofs draining directly to the sewers, as previously explained, the McMath formula indicates a somewhat larger run-off for areas up to about 12 acres, and a smaller run-off for larger areas, the difference approximating 30 per cent. when the tributary drainage area reaches 60 acres.

TABLE 91.—COMPARISON OF RATIONAL METHOD OF SEWER DESIGN WITH MCMATH FORMULA, FOR ST. LOUIS CONDITIONS (SEE FIG. 112 AND TABLE 90)

Number	Between man-holes	Drainage area, acres[1]	Maximum discharge in cu. ft. per second: By rational method	Maximum discharge in cu. ft. per second: By McMath formula	S, per cent.	Diameter of sewer, inches[2]: By rational method	Diameter of sewer, inches[2]: By McMath formula
1	177–176	0.88	1.94	3.22	0.50	12	12
2	176–175	1.64	3.61	5.25	0.50	15	15
3	175–165	3.05	6.71	8.60	1.00	15	18
4	166–165	2.12	4.67	6.46	1.50	12	15
5	165–164	4.26	9.39	11.3	1.50	18	18
6	164–163	7.26	15.97	17.4	3.00	18	18
7	172–173	1.13	2.49	3.92	0.50	12	15
8	173–174	1.79	3.94	5.58	0.50	15	15
9	174–163	3.13	6.89	9.80	3.00	12	15
10	163–162	12.10	26.64	26.0	1.00	24	24
11	162–161	14.86	32.70	30.80	1.00	27	27
12	171–170	1.86	4.09	5.82	1.50	12	12
13	170–169	2.61	5.75	7.60	0.75	15	15
14	169–161	3.36	7.40	9.30	2.00	15	15
15	161–160	18.72	41.2	37.2	3.25	24	24

[1] Including allowance for roofs draining directly to sewers.

[2] Using Horner's curves for sewer sizes.

TABLE 91.—COMPARISON OF RATIONAL METHOD OF SEWER DESIGN WITH McMATH FORMULA, FOR ST. LOUIS CONDITIONS.—(*Continued*)

Section	Between manholes	Drainage area, acres[1]	Maximum discharge in cu. ft. per second		S, per cent.	Diameter of sewer, inches[2]	
			By rational method	By McMath formula		By rational method	By McMath formula
16	160–159	19.83	43.6	39.0	2.50	27	24
17	131–194	0.82	1.88	3.05	2.00	12	12
18	194–197	2.77	6.10	8.00	0.60	18	18
19	197–257	2.77	6.10	8.00	2.00	12	15
20	257–159	5.55	12.20	13.9	5.00	15	15
21	159–*a*	28.07	61.8	51.3	2.00	26×39	24 × 36
22	168–167	2.36	5.20	7.03	4.75	12	12
23	167–*a*	8.36	18.4	19.4	29.30	12	12
24	*a*–157	32.73	72.0	58.3	2.56	26 × 39	24 × 36
25	157–*b*	33.33	73.3	59.0	1.50	30 × 45	28 × 42
26	*b*–*c*	45.48	100.0	75.8	1.00	34 × 51	32 × 48
27	274–275	0.54	1.19	2.17	1.50	12	12
28	275–270	4.08	8.98	10.9	0.80	18	18
29	365–270	0.87	1.95	3.19	2.00	12	12
30	276–270	1.12	2.46	3.89	1.25	12	12
31	270–269	13.14	28.9	27.9	6.00	18	18
32	269–*c*	14.36	31.6	30.0	16.70	18	18
33	*c*–155	48.26	105.0	79.5	1.00	36 × 54	32 × 48
34	271–272	0.53	1.19	2.14	2.50	12	12
35	272–155	1.71	3.76	5.45	4.75	12	12
36	273–277	0.82	1.81	3.05	1.50	12	12
37	277–155	2.66	5.85	7.75	1.00	15	15
38	155–*d*	54.36	117.0	87.5	1.00	36 × 54	34 × 51
39	*d*–154	57.10	122.0	91.0	1.00	36 × 54	34 × 51
40	154–*e*	57.29	122.0	91.5	1.00	36 × 54	34 × 51
41	*e*–153	59.71	127.0	94.5	1.00	36 × 54	34 × 51
42	153–136	61.34	129.0	97.0	1.00	36 × 54	34 × 51

[1] Includes roof water reaching sewers. [2] By Fig. 112.

It is interesting to note that in this comparison the difference in the resulting size of sewer is comparatively insignificant. It does not follow, however, that this would be the case in other comparisons. Much would depend upon the judgment exercised in selecting coefficients, particularly in the application of the rational method.

ADDITIVE METHOD OF COMPUTING RUN-OFF

This method, developed by Carl H. Nordell (*Engineering News*, March 11, 1909) and used in the design of storm-water sewers in the

Borough of Queens, New York City, is merely a modification of the rational method. In its application a storm of constantly varying intensity is assumed, and a curve showing the intensity of precipitation at each instant after the beginning of the storm is constructed. Then for any elementary area, the greatest run-off will be produced by the portion of the storm, equal in duration to the "time-length" or time of concentration of the area, during which the average intensity of precipitation was greatest. For a large area, composed of a number of elementary areas, the maximum rate of flow occurs when some of the elementary areas are contributing less than their maximum discharge. The average rate of rainfall applicable to each elementary area is obtained from the storm curve by taking an ordinate at a point above or below that giving the largest maximum for the controlling (usually the largest) elementary area, equal to the time required for water to flow in the sewer between these two areas.

The general principle underlying this method is sound. The assumption that the rate of rainfall is constantly changing, requiring the use of average rates of precipitation, is erroneous, however. As a matter of fact, in many storms the rates are substantially uniform for moderate periods of time; and for the maximum rates, lasting shorter periods of time, the variations in rate during those periods are insignificant.

The difficulty in assuming a suitable curve for distribution of intensities is a very real one, and involves uncertainties much greater than those involved in the application of the rational method in the usual way.

BASIS OF DESIGN OF STORM-WATER DRAINS IN VARIOUS CITIES

Baltimore.—The rational method is employed, using the rainfall curve, $i = 300/(t + 25)$. As a preliminary step and to fix approximately the value of t, a computation of the run-off is made by the McMath formula, using $C = 0.75$ and $i = 3$, from which the size of the drain and the corresponding velocity of flow are tentatively computed. From this velocity the value of t is estimated, and the rational method is applied in the usual way.

Boston.—The rational method is employed, using Dorr's rainfall formula, $i = 150/(t + 30)$ and coefficients varying from 0.15 to 0.90. Coefficients are selected to apply as nearly as may be estimated to the conditions the districts may attain in 25 or 30 years.

Cambridge, Mass.—Computations are made by Bürkli-Ziegler formula, using a coefficient of 0.50 to 0.60, and a rainfall rate of 1.50 in. per hour. Many measurements have been made, which show that the crest of the wave follows the greatest intensity of rain in about 20 minutes for

drainage areas of 200 to 300 acres, and that the intensity of precipitation for a 20-minute period is about 2.50.

Cincinnati.—The rational method is employed, with the following rainfall curve, $i = 16/t^{0.5}$. The coefficients of run-off used vary from 0.2 to 0.9, according to the assumed development of the territory at the end of a period of 40 years.

Cleveland.—Robert Hoffman, Chief Eng., Department of Public Service, states that present practice in Cleveland is to first compute the time required for water to reach various points in the system, and then from curves based upon intensity of rainfall, read directly the quantity of water to be cared for. The curve for total run-off (coefficient $C = 1$) is

$$q = i = \frac{5040}{t^2 + 1440} \text{ or } Q = \frac{5040A}{t^2 + 1440}$$

Other curves are drawn, corresponding to $C = 0.5$ at the beginning of the storm and 0.7 after 1 hour; 0.4 at the beginning and 0.6 after 1 hour; and 0.3 at the beginning and 0.55 after 1 hour. Their equations are:

$$q = \frac{4200}{t^2 + 2400}, \quad q = \frac{3780}{t^2 + 2700}, \quad q = \frac{4158}{t^2 + 3960}$$

Of the first curve, that for intensity or rate of run-off per acre when $C = 1.0$, Hoffman says: "With the exceptions of a few storms, the rain rate curve amply provides for such storms as occur in this section." It has been taken as a reasonable basis for design. By interpolating between the curves given, such coefficient of run-off may be employed as the judgment dictates. It should not be forgotten that modification of the rainfall curve by applying a variable coefficient assumes that the greatest intensity is at the beginning of the storm, and that the intensity decreases regularly as the storm progresses.

Louisville.—Quantities of storm water are estimated by means of the McMath formula, using a rainfall rate of 2.25 and run-off coefficients varying from 0.4 to 1 and even more. The slope, S, is taken as 4 per 1000 where the district is very flat and is increased proportionately in steeper districts in the eastern part of the city.

Newark, N. J.—The Hering formula is used, assuming $i = 1.5$, and the following values for C; suburban areas, 1; well-developed areas, 1.25; completely developed areas, 1.50.

New York City, Borough of the Bronx.—The same method is employed as in the Borough of Richmond. For intensity of precipitation the formula used is $i = 120/(t + 20)$ and C is taken as between 0.14 and 0.75.

New York City, Borough of Brooklyn.—McMath's formula is used, assuming a maximum rate of rainfall of 3 in. per hour for 30 minutes. C is taken between 0.50 and 0.75.

New York City, Manhattan Borough.—The quantities of storm water are estimated by the Hering formula,

$$Q = CiA^{0.85}\ S^{0.27}$$

Ci is taken as 1.02 for suburban districts; 1.39 for well-developed districts, and 1.64 for completely built-up areas. The corresponding values of C are from 0.50 to 0.80.

New York City, Borough of Queens.—The additive method is employed. The rainfall curve employed is shown by the equation

$$i = \frac{72.5t - 299.85}{t\ (t + 4.14)}$$

and this is assumed to follow a 10-minute rainfall at the rate of 3 in. per hour. C is assumed between 0.30 and 0.81.

New York City, Borough of Richmond.—Quantities are estimated from the formula $Q = CiA$.

The rainfall equation $i = 105/(t + 25)$ is used and this precipitation is assumed to follow a 5-minute rain at the rate of 3.5 in. per hour. C is taken as ranging between 0.36 and 0.82.

New Orleans.—Run-off curves based on the Bürkli-Ziegler formula are used. Maximum rainfall rate of 6 in. per hour for short periods is assumed, and the following run-off factors; for densely built-up areas, $C = 0.80$; for medium conditions, $C = 0.60$; for sparsely built districts $C = 0.50$; for rural conditions $C = 0.20$.

Pawtucket, R. I.—Sewers designed about 1885 by the Bürkli-Ziegler formula, with 2-in. rate of rainfall, have in many cases proved too small. The rational formula is now used, having a rainfall curve constructed from local observations, and taking the rate of precipitation corresponding to a period of 8 minutes plus one-half the time of flow in sewer. The coefficient is varied to suit the conditions in the judgment of the engineer.

Providence.—City Engineer Otis F. Clapp states in a letter to the writers that the ordinary combined sewers of Providence are designed to care for 0.5 in. of rain per hour and as a usual thing have proved satisfactory. Storm-water drains proper, however, are designed to receive from 0.75 to 2 in. per hour, according to location and conditions.

St. Louis.—The rational method is employed, as explained in detail earlier in this chapter. For intensity of precipitation the equation $i = 56/(t + 5)^{0.85}$ is employed, and values of C ranging from 0.20 to 0.95 are used, depending upon the character of the surface.

Worcester, Mass.—The Bürkli-Ziegler formula is employed, using a rainfall rate of about 1 in. (varying somewhat with size of area and slope of surface), and coefficient ranging from 0.62 to 0.75.

FOR HOW SEVERE STORMS SHOULD STORM DRAINS BE DESIGNED

From an economic point of view, it is possible to compute approximately the point beyond which it is more economical to allow overflowing and to pay or suffer the damages rather than to increase the size of storm-water drains, if it is possible to estimate satisfactorily the damage which may result from flooding, and if information is available to indicate the relative frequency of storms of various degrees of severity.

Practically, however, such computations are of little significance. Local circumstances and conditions, physical and financial, have usually a controlling effect upon the extent to which such drains can be designed to care for extreme maximum rainfalls. The legal responsibility of the community is also an important consideration, although it should not be controlling, since any damage from overflowing must be suffered by members of the community, if not by the entire community as a municipality.

The responsibility of a city for damages of this kind is generally held to depend upon the character of the storm, and the courts have held that "rainfalls are differentiated for judicial purposes into ordinary, extraordinary and unprecedented classes. Ordinary rain storms are those which frequently occur, extraordinary storms are those which may be reasonably anticipated once in a while, and unprecedented storms are those exceeding any of which a reliable record is extant. The usual rule in determining the responsibility of a city was stated many years ago by the New York Court of Appeals, 32 N. Y. 489, as follows: 'If the city provides drains and gutters of sufficient size to carry off in safety the ordinary rainfall, or the ordinary flow of surface water, occasioned by the storms which are liable to occur in this climate and country, it is all the law should require.'" (*Eng. Rec.*, June 8, 1912).

The question of what constitutes "ordinary" storms still remains. Are storms which may reasonably be expected to occur on an average once in 10 years ordinary or extraordinary? There seems to be no way of satisfactorily answering this question, and it will be necessary for the engineer to decide in each case what is the reasonable condition to be met. The abstracts of legal decisions quoted upon the following pages may be of assistance in guiding the judgment, particularly with respect to the legal responsibility of a municipality for flooding due to inadequate storm-water sewers.

Generally much greater damage will result from the flooding of a main or trunk sewer than from the inadequacy of a branch drain; yet the damage from overflowing upon a large number of branches may be much more serious than would result from flooding of the main sewer,

particularly if storm relief overflows can be provided. Moreover, it is a much simpler and usually a less expensive operation to reinforce or duplicate a main sewer than to rebuild many small laterals. The additional cost of constructing the latter of ample size when first built will generally be inconsiderable, while the additional cost of a main sewer large enough to care for the most severe storms may be prohibitive, particularly if it is to be designed to meet future conditions, which may not exist for many years to come. It is therefore generally advisable to build branch or lateral sewers of the capacity which will ultimately be required, giving the mains and submains a capacity sufficient for present conditions and to provide for the growth for some years, with the expectation that new relief sewers will ultimately be required to care adequately for the entire run-off from the district.

ABSTRACTS OF LEGAL DECISIONS RELATING TO FLOODING OF SEWERS

(Taken from "American Digest, Municipal Corporations")

Alabama, 1902.—A city for the efficiency of its sewers is bound to make provision for such floods as may be reasonably expected from previous occurrences, though at irregular and wide intervals of time. (Arndt *vs.* City of Cullman; 31 So. 478; 132 Ala., 540.)

Delaware, 1888.—In an action for damages to property from an overflow of a sewer during a severe storm caused, as alleged, by the insufficiency of the sewer and an obstruction in it, it is for the jury to determine whether the injury was caused by the insufficiency of the sewer or any obstruction in it owing to the neglect of the city, or by the magnitude of the storm, discharging a greater quantity of water than might reasonably be expected from past experience. (Harrigan *vs.* City of Wilmington; 8 Houst., 140; 12 Atl., 779.)

Delaware, 1893.—A city is not liable for damages caused by backwater from a sewer caused by an excessive and phenomenal rainfall against which the city could not reasonably be bound to provide. (Hession *vs.* City of Wilmington; 40 A 749.)

Delaware, 1893.—The testimony of an engineer as to the necessary capacity of a sewer in a particular locality for ordinary occasions, is proper evidence of what is an extraordinary rainfall. (Hession *vs.* City of Wilmington; 27 Atl., 830.)

Illinois, 1897.—Where a city has provided sewers or drains of ample capacity to carry off all water likely to fall or accumulate upon the streets on all ordinary occasions, it is not guilty of negligence in failure to anticipate and provide for unanticipated and extraordinary storms. (City of Peoria *vs.* Adams; 72 Ill., App. 662.)

Illinois, 1901.—A municipal corporation must see to it that the outlet of its sewers is of ample capacity to carry off all the water likely to be in it, but it is not liable for damages caused by an extraordinary and excessive rainfall. (City of Chicago *vs.* Rustin; 99 Ill., App. 47.)

Iowa, 1896.—The fact that a city has notice that drains constructed by it to carry off street surface water are insufficient, fails to use ordinary diligence to make such changes as appear reasonably necessary to make the drains serve the purpose intended, does not render the city liable for the resulting overflow of private property where it did not accelerate the flow of the water, or collect the same and discharge it on such property otherwise than it would naturally have been discharged thereon, and it was not negligent either in devising or in adopting the plans of the drains. (Knostman & Petersen Furniture Co. *vs.* City of Davenport; 99 Iowa 589.)

Kentucky, 1881.—A municipal corporation is responsible for damages caused by the want of due care and skill in constructing a sewer, and also for the insufficient size or capacity thereof. (City of Covington *vs.* Glennon; 2 Ky. Law Rep., 215.)

Massachusetts, 1903.—Where, in an action against a city for damages arising from water coming on plaintiff's premises through a city sewer, there was no evidence that the sewers were defective in construction or obstructed or out of repair and nothing to show that they were established otherwise than by persons acting as public officers under the statute, and the proof tended to show that the defect, if any, in the sewers was in the system which failed to carry off immediately a great accumulation of water due to a heavy rainfall, plaintiff could not recover. (Manning *vs.* City of Springfield, 184 Mass., 245.)

Minnesota, 1897.—A city which, in grading a street, constructed an embankment across a stream, making a culvert for the water to pass through, cannot be held liable for damage caused by the insufficiency of such culvert to carry off the water during an unusual storm, unless such insufficiency resulted from a failure to use ordinary care or skill in its construction. It was not required to anticipate such storms as from the history of the country would not reasonably be expected to occur, and if it employed competent engineers who were justified in believing and did believe that the culvert was of sufficient size, it was not negligent. (Taubert *vs.* City of St. Paul, Minn.; 68 Minn., 519; 71 N. W., 664.)

Missouri, 1894.—Where the negligence of a city in failing to keep its sewers open contributed to the damage to property, it is liable although the rain causing the damage was of an extraordinary character. (Woods *vs.* City of Kansas; 58 Mo. App. 272.)

Missouri, 1901.—Where a city set up the defense that the breaking of a sewer was caused by the act of God manifested in an unusual rainfall, and there was no evidence that the sewer was defective by reason

of improper construction and failure to repair, and that the rainfall was of an unusual character, it was proper to charge that if an unusual rainfall would have caused the breaking of the sewer notwithstanding its defect then the city was not chargeable with negligence; but if the breaking was caused by such defects or if it was caused by such defects commingled and concurring with unusual rainfall then the city was liable. (Brash *vs.* City of St. Louis; 161 Mo., 433.)

Missouri, 1903.—Where a rainstorm such as had never occurred before, caused a flooding of the lands from a sewer, no greater than would have occurred under natural conditions, the sewer having been scientifically built according to the best judgment of the engineers and having a sufficient capacity under ordinary conditions, the injury results from an act of God, for which the city is not liable. (Gulath *vs.* City of St. Louis, 179 Mo., 38.)

New York, 1861.—There is something very like a contract to be implied from the construction of a sewer at the expense of the adjacent property, that it may be used to drain the property thus charged with its construction, and it would seem that the adjacent property holders have a right to open drains into it; and in a suit by such adjacent property holder who had opened his drain into the sewer upon his own responsibility and whose premises had been flowed by backwater through the drain in a freshet, it was held that a verdict giving him damages must be sustained. (Barton *vs.* City of Syracuse, N. Y.; 37 Barb. 292 affirmed (1867); 36 N. Y., 54.)

New York.—A sewer in the city of Troy, built by the owners of land through which it passed and by the city where it passed through its streets and alleys, passed through the premises of plaintiff and emptied into the Hudson river. Another sewer built by the city was connected with it. One T. petitioned the common council for leave to enlarge the opening between the sewers. This was referred to a committee with power. The city commissioner, whose duty it was to look after and inspect sewers, authorized the change to be made. In doing the work T. built a wall across the sewer first mentioned, partially obstructing the outlet, and diminishing the capacity of the sewer, by reason whereof it became clogged and filled up and a storm occurring the accumulated water burst open the sewer upon plaintiff's premises, causing damage. T. presented his bill for the work to the common council, which was audited and paid. Held, that the city was chargeable with notice of the obstruction and was liable for the damages resulting therefrom. (Nims *vs.* City of Troy; 59 N. Y., 500.)

New York, 1902.—Where a municipality has constructed and maintained a sewer adequate for all ordinary purposes it is not liable for injuries to abutting owners, caused by overflow of the sewer due to a

storm of extraordinary violence. (Sundheimer *vs.* City of New York; 79 N. Y. S., 278; 77 App. Div. 53, reversed 1903.)

Pennsylvania, 1882.—In an action against a municipal corporation for damages for injuries sustained by the bursting of a sewer, owing to its negligent construction by defendant, when it appeared that owing to an extraordinary flood the breakage would have happened even if the negligence complained of had not existed, no damages can be recovered. (Bolster *vs.* City of Pittsburgh; Leg. J 204.)

Pennsylvania, 1902.—The mere omission of municipal authorities to provide adequate mains to carry off the water which storms and the natural formation of the ground throw on city lots and streets, will not sustain an action by an owner of land, against the municipality, for damages arising from the accumulation of water. (Cooper *vs.* City of Scranton; 21 Pa. Super. Ct. 17.)

Texas, 1894.—Where a lot owner knows that his premises will be flooded in case of a heavy rain, unless a certain city drainpipe in the street adjacent thereto is cleaned out, and gives no notice of it and makes no effort to remedy the defect, he cannot recover of the city, damages caused by flooding his premises during such storm. (Parker *vs.* City of Laredo; 9 Tex. Civ. App. 221; 28 S. W., 1048.)

West Virginia, 1896.—A city is not bound to furnish drains or sewers to relieve a lot of its surface water. (Jordan *vs.* City of Benwood, W. Va.; 42 W. Va., 312.)

CONCLUSION

While the problem of determining the quantity of storm water to be carried by drains is still difficult and indeterminate, much advance has been made during recent years in the methods of attacking it. This has been due largely to the securing of accurate records of rainfall, showing duration and intensity. More information showing the actual run-off from rains of known intensity upon areas carefully studied to determine their local characteristics (similar to that given in the next chapter), and observations of the time required for the water to reach the sewers, is very much needed, particularly to assist the engineer in making a judicious selection of the coefficient of run-off. There is also need for detailed and long-continued studies of the distribution of rainfall within areas of comparatively small extent, say up to 5 square miles, in order to furnish definite information relating to the area covered by heavy storms, and the rate of diminution of intensity of precipitation as the distance from the center increases. The older empirical formulas are gradually giving way to rational methods of computation, which enable the engineer to exercise his judgment more readily and design structures peculiarly adapted to local conditions.

CHAPTER IX

GAGING STORM-WATER FLOW IN SEWERS

Methods of gaging the flow in sewers have been referred to in Chapter II upon Measurements of Water. As a general rule, weirs, current meters, or other measuring devices are impossible of employment in gaging flood flows, and recourse must be had to computation of the discharge from observations of the depth in the sewers, and from the known or measured slope and known or assumed conditions, such as roughness, affecting the flow.

As storms are likely to occur at any time, and observers cannot be constantly on duty, automatic recording gages for showing the depth of sewage at any moment are practically indispensable. At least two of these are necessary, in order to determine the slope of the water surface, which is frequently very different from the slope of the sewer. In addition to the depth gages, it is desirable to have a number of maximum flow gages, which show the greatest depth of sewage at the point of installation since the last observation, but give no further information.

There are several kinds of automatic recording gages for showing the elevation of the sewage or water level at a gaging point, but all of them belong to one or the other of two general types, float gages and pneumatic pressure gages. Recorders of either type are also applicable to weirs where a continuous record of the head upon the weir is desirable, or to any case when an autographic record of the elevation of a water surface is required.

As noted in Chapter VI, upon Precipitation, it is extremely important that every automatic gage should have a good clock movement, that it should be regulated to keep correct time, and synchronized with the clocks of all other gages the records of which may be studied jointly. If the clocks were furnished with dials the regulation and synchronizing would be greatly simplified, but few of the gages now on the market have such dials. It may well be questioned whether electrical operation of the clocks on large works where several gages are employed would not be practicable, as it certainly would be desirable.

FLOAT GAGES

In gages of this type, a float contained in a pipe or other suitable guide in which the sewage stands at the same height as in the sewer,

is connected with a recording apparatus through the medium of a cord, chain, tape, or by a stiff rod or tube.

The Hydro-chronograph.—This is made by the Hydro Manufacturing Co. of Philadelphia, and consists of a float and a recorder, Fig. 114. The float is connected by a chain with a sprocket wheel at the recorder. The motion of the float is thus transmitted, on a reduced scale, to a pen moving in front of a vertical recording drum, which is rotated monthly, weekly, or daily, as desired. The diameter of the cylinder is such that a time scale of about 1 in. per hour may be employed.

The amount of reduction in vertical scale will depend upon the range of motion of the float. This company manufactures a weir gage in

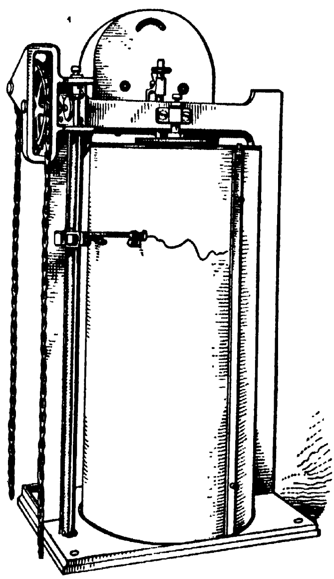


FIG. 114.—The Hydro-chronograph.

which the fluctuations of water level are recorded without reduction; but this can be employed only for a range of about 2 ft. For large sewers it is impracticable to use a drum long enough to cover the range of elevation without reduction. The list prices of these instruments range from $100 to about $200.

Friez's Automatic Water Stage Register.—This instrument is made by Julien P. Friez of Baltimore. As shown in Fig. 115 motion of the float causes the drum to rotate, while the pen is caused to move parallel

to the axis of the drum, by means of clockwork. The clock is provided with a dial and hands, which facilitates greatly the proper regulation of the timepiece.

The cylinder is 8 in. long and 12 in. in circumference. The clock can be arranged to drive the pen the length of the cylinder once a week or once a day. In the latter case, the time scale would be 1/3 in. to the hour, and this is the largest scale for which the instrument is regularly

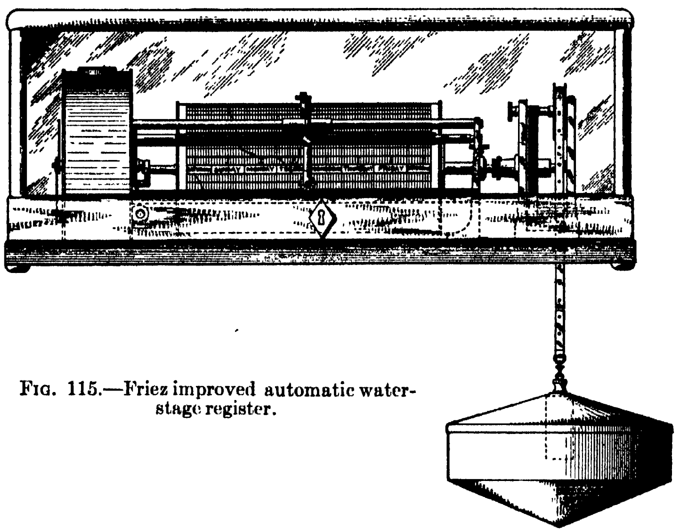


FIG. 115.—Friez improved automatic water-stage register.

made. Sprocket wheels are provided for different ranges in height of water, as follows:

Range of water level	Total width (circumference) of chart	Scale of heights
1 ft.	1 ft.	1 ft. to 1 ft.
5 ft.	1 ft.	0.2 ft. to 1 ft.
10 ft.	1 ft.	0.1 ft. to 1 ft.
15 ft.	1 ft.	0.0667 ft. to 1 ft.
20 ft.	1 ft.	0.05 ft. to 1 ft.

List prices of these instruments range from $115 to $150.

Builders Iron Foundry Water Level Recorder.—In this gage the cord from the float moves an arm carrying a pen in front of a circular chart, which is rotated by clockwork, Fig. 116. The pen accordingly moves in a circular arc, and the time-scale varies with the position of the pen. The instrument is enclosed in a cast-iron box mounted upon a hollow stand-

ard through which the float cord passes. It is made in two sizes, having 8-in. and 12.-in. dials, and the prices are $75 and $90, respectively, or they can be obtained with an iron outer door for $5 additional.

Obviously, with this gage, the scale of heights as recorded upon the chart will depend upon the range to be covered and the size of the chart. A rectangular chart is not necessary for records of this kind, and the only disadvantage of this form of record is that the time-scale is unduly small when the pen is in its lowest position.

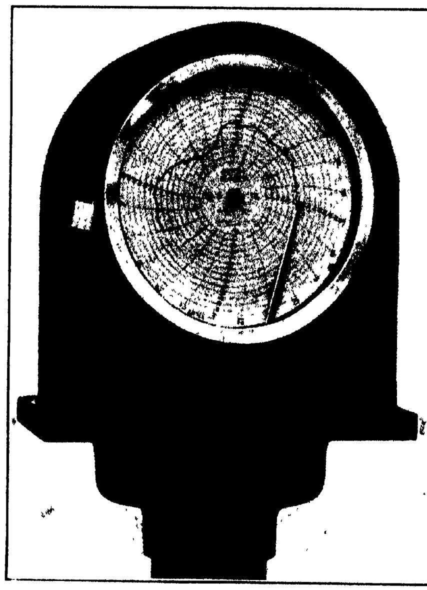


FIG. 116.—Water-level recorder (Builders Iron Foundry).

Builders Iron Foundry has also in some cases constructed a modification of the recording instrument of the Venturi meter for use with a float, to indicate and record directly the rate of flow over a weir, and also to integrate these rates and show on a recorder the total quantity passed.

Stevens Continuous Water-stage Recorder.—In this instrument the pencil is moved horizontally by a belt controlled by a wheel over which a cord from a float passes. The record is made upon a horizontal cylinder around which a sheet of paper is made to revolve, being reeled off from one roll and on to another, so that it is possible to use

very long sheets of paper and keep continuous records for long periods of time without renewing the record sheet. A mechanism is also provided by which the motion of the pencil carriage is reversed after reaching the limit of its motion in one direction, and in this way it is possible to record an unlimited range in elevation without reduction of the scale.

As ordinarly constructed, the cylinder of the Stevens gage is driven by a weight, and it is claimed that with this method of operation the recorder can be left from one to two months without attention, and that in certain cases records of 95 days have been made without attention to the instrument. It is also constructed with a spring for driving the cylinder, but in this case it is necessary to wind the spring about once in 8 days.

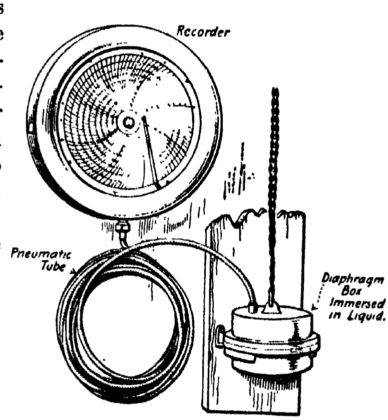


FIG. 117.—Diaphragm pressure gage.

These instruments are made by Leupold & Voelpel of Portland, Ore. The list price of the weight-driven instrument, including copper float, counterpoise, and one year's supply of paper, is $135. The base of the instrument measures 10 × 12 in., the cover is 8 in. high, and the total weight, including float and weights, is about 65 lb., packed for shipment.

Pneumatic Pressure Gages.—In these gages a diaphragm box or pressure chamber is immersed in the liquid, and the changes in pressure resulting from the rising or falling surface are transmitted through a small pneumatic tube to a recording apparatus located at any convenient point.

Fig. 117 shows an instrument of the diaphragm type. It is made for either 8-in. or 12-in. charts, and the prices range from $55 to $80, including 25 ft. of connecting tubing. These instruments are made by the Industrial Instrument Co. of Foxboro, Mass., and by The Bristol Company of Waterbury, Conn.

The Sanborn Flow Recorder, Fig. 118, made by the American Steam Gage and Valve Manufacturing Co., Boston, may be placed in a manhole, at the sidewalk, or in a near-by building. One-fourth inch copper tubing connects from the recorder to the inlet at the sewer where is located a "compensator," which is a special form of diving-bell. It

resembles a piece of tubing, 1-1/2 in. in diameter varying in length from a few inches for small sewers to 3 ft. for 20-ft. sewers; it is placed slanting, on an angle of 45 deg. with the vertical, in the direction of flow, and extends to within a few inches of the bottom of the sewer. This compensator is constructed smooth outside and inside so that sewage is not apt to collect. The inlet is at the very bottom. Claims made for this device are, that no float is required, no diaphragm at the inlet, the pressure medium is air and will not freeze, and the recorder

FIG. 118.—Sanborn flow recorder.

may be placed in any convenient location. The price complete, with compensator and 25 ft. of tubing, is $75.

Other Gages.—For very accurate determinations of water levels, float or pressure gages are not applicable on account of friction in the moving parts, back-lash of mechanical parts, etc. On account of capillarity, ordinary staff gages inserted directly in the water, and having scales marked on them with zeros set at some determined elevation, are also uncertain. An improvement on the ordinary staff gage consists in the use of a plumb bob suspended by a fine wire, which passes over a wheel at the end of an arm held horizontally over the

water. By a suitable scale marked on the horizontal arm, readings are obtained by lowering the plumb bob until it just touches the water. When the determinations have to be made in dark or inaccessible places, so that the point of the bob cannot be seen, an electrical contact may be brought into use. In this arrangement one pole of a battery is connected to the wire carrying the plumb bob, while the other pole is connected through a delicate galvanometer to an iron cylinder surrounding the plumb bob and inserted in the water, thus forming a "stilling box." When the plumb bob touches the surface of the water, sufficient current passes to deflect the galvanometer which is placed at some convenient point near the scale board (*Eng. Rec.*, 1913, p. 192).

The most accurate gage is, however, the hook gage invented by Boyden about 1840. This takes advantage of the surface tension of the water surface, and consists, as the name implies, of a hook attached to a rod carrying a scale, which may be moved up and down or clamped to a supporting contrivance provided with a slow motion arrangement and vernier. The gage is operated by lowering the rod until the point of the hook is below the water surface; the rod is then raised slowly until a protuberance on the water surface is noted just over the point of the hook. The point of the hook does not break the surface of the water immediately, but carries the surface film up with it and the beginning of this phenomenon can be very accurately noted by watching the reflected light upon the surface. In a good light, with suitable verniers, differences as small as 0.0002 ft. can be determined.

A gage of this character, known as the Boyden hook gage, is on the market. This gage has a frame of wood 3 ft. long by 4 in. wide, in a rectangular groove of which is made to slide another piece carrying a metallic scale graduated in feet and hundredths, from 0 to 2 ft. Connected with the scale is a brass screw passing through a socket fastened to another sliding piece, which can be clamped at any point upon the frame, and the scale with hook moved in either direction by the milled nut or slow motion screw. The scale is provided with a vernier which enables the setting to be read to thousandths of a foot.

This gage has a number of disadvantages, particularly when used in connection with sewer gagings. The most serious are: 1. The material is largely wood, which is objectionable for permanent installations in damp places. 2. The zero and vernier of the gage are at one end of the instrument, while the slow motion screw is at the other end, thus making it very awkward in operation.

A much more satisfactory type of hook gage is the Emerson gage, illustrated in Church's "Hydraulic Motors." This instrument is accurate, durable, and convenient to use, but is heavy, not very portable, and decidedly expensive.

Acting on suggestions from Metcalf & Eddy, W. & L. E. Gurley have

placed on the market the hook gage shown in Fig. 119. This is constructed wholly of non-corrosive metal, is light, strong and has an adjustable hook.

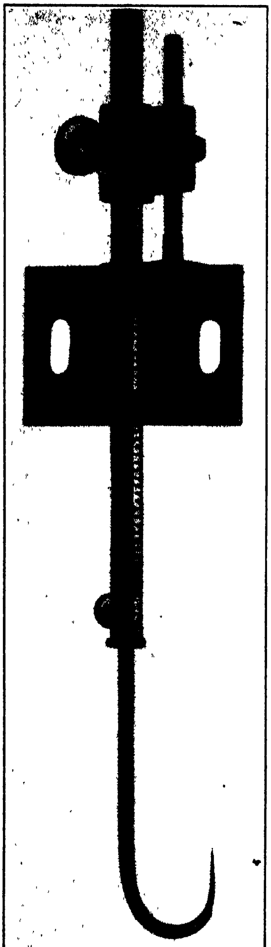


FIG. 119.—Hook gage suggested by authors (Gurley).

Setting Sewer Gages (Water Level Recorders).—Local conditions will determine the locations of the points at which gages should be established. It is, however, always necessary that the sewer for a considerable distance upstream and for a less distance below the gage should be in such condition that the quantity flowing can be computed from the depth in the sewer. The cross-section and slope must be uniform, there must be no curves, and no inlets or obstructions to cause disturbance in the flow, and the condition of the interior should be known so that a coefficient of roughness can be applied with a good degree of accuracy. Moreover, the velocity of flow in the sewer should not be great. In order to be sure of the results, it is necessary to have gages at each end of such a stretch of sewer, to determine the slope of the water surface.

Gaging apparatus should be installed in a separate chamber or gaging manhole at one side of the sewer, to protect the instruments and make them easily accessible for observation or adjustment. This chamber should be connected with the sewer so that water will stand in the chamber at the level of the sewage in the sewer. It is desirable to have a small flow of clean water into and through the gaging chamber and thence to the sewer, in order that the liquid surrounding the instruments or floats shall be water and not sewage, thus avoiding clogging and derangement, as well as rendering the chamber a much pleasanter place to enter than it would be if partly filled with stagnant sewage.

An excellent example of a special gaging chamber or manhole is

shown in Fig. 120, an illustration of the chamber constructed by the sewer division of the City of Cincinnati. This is arranged for a gage of the diaphragm type, from which the pressure pipe will be conducted through a wrought-iron pipe to an iron box mounted at the curb

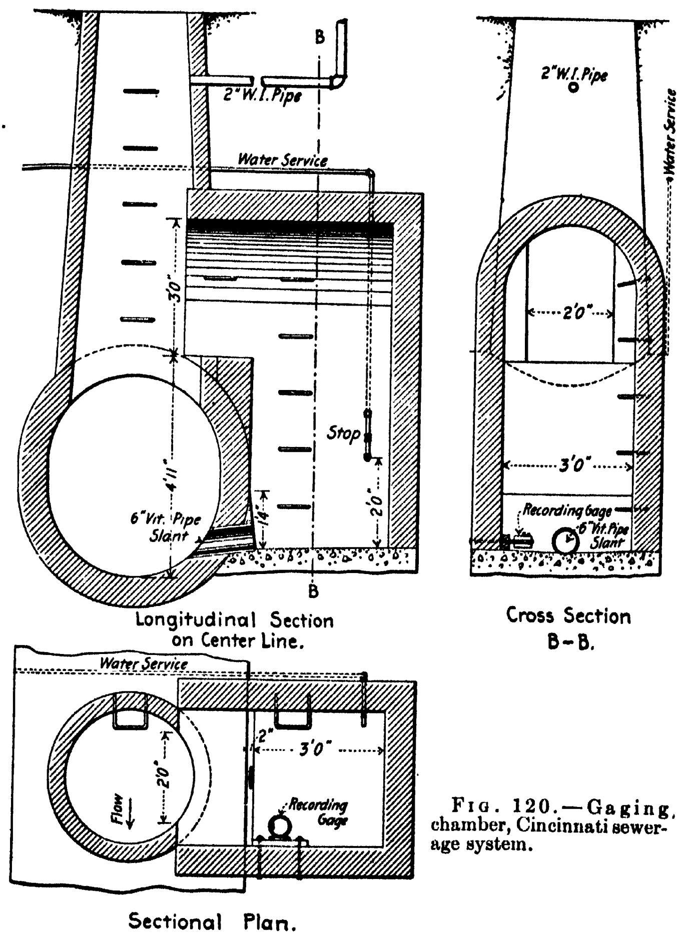


FIG. 120.—Gaging chamber, Cincinnati sewerage system.

or a recorder within a house. If a float gage were to be used, it would be necessary to locate the recorder within the chamber or upon a post or in a building directly over it. The location within the chamber is

objectionable, owing to the rapid corrosion of the clockwork and other parts of the instrument, as well as to the effects of moisture upon the paper chart. Where the sewer is in a street it is usually not possible to locate a building or even an iron post and box directly over the float chamber, unless the latter is extended so as to lie at least partly under the sidewalk. While the design of the gaging chamber is good, the connection between it and the sewer may be criticized, because it is not normal to the inner surface of the sewer, and therefore the liquid in the chamber may sometimes not stand at the same level as in the sewer.

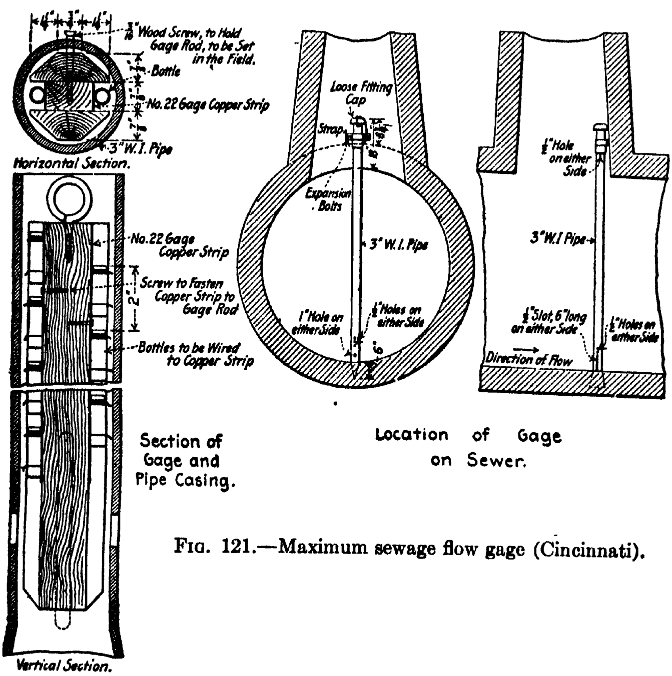


FIG. 121.—Maximum sewage flow gage (Cincinnati).

Maximum Flow Gages.—In its simplest form, a maximum flow gage consists of a strip of wood held securely in a sewer in a vertical position, and so coated that it will show at a glance how much of the strip has been submerged at any time since it was put in place.

The first maximum gages consisted simply of strips of wood coated with whitewash, and fastened firmly into the manholes. Later, to make more certain the determination of the point to which the sewage

had risen, sand was imbedded in the whitewash, and mucilage was used to help retain the sand, it being supposed that the sand would fall away from the portions which had been wet. It has been found, however, that sometimes the moisture in the atmosphere would cause the whitewash and sand to disappear from the strip where it had not been submerged, and sometimes they would continue to adhere in spite of immersion.

The illustration, Fig. 121, shows a maximum flow gage which has been devised by the sewerage engineers of the City of Cincinnati, H. S. Morse, Engineer in Charge, to overcome these defects. It will be noticed that to clamp the rod firmly in place it is put inside a 3-in. steel pipe secured in an upright position, with openings near the bottom to allow the sewage in the pipe to rise and fall with that in the sewer. The rod itself is supported by a wood screw held by a "bayonet joint," a slot in the pipe having a right-angled change of direction. The rod itself carries a series of vials so fastened that their mouths are 1 in. apart vertically. It is therefore obvious that the sewage has been at least as high as the highest vial which is found filled; and it is only necessary to invert the rod and empty the bottles, and then replace the rod, to have the gage ready for another observation. This gage has proved satisfactory under ordinary conditions, but the results were not satisfactory when velocities greater than 8 ft. per second were encountered. A similar gage has been used at Pawtucket, R. I., by George A. Carpenter, city engineer.

In Boston, Mass., maximum sewer gagings are recorded by a circular box float, with a central opening through which a vertical guide rod runs. The fit is a very loose one, but a pair of sheet brass springs attached to the float press lightly on the guide and hold the float at the highest elevation to which it is lifted.

ACTUAL MEASUREMENTS OF STORM-WATER FLOW

It must be admitted that the determination of the run-off factor from actual gagings is extremely unsatisfactory. Only a limited number of such gagings have been made, and even the best of these leave much to be desired, and the coefficients deduced from them can be considered only as approximations. Nevertheless, these measurements are of much importance, not only because they furnish the only experimental determinations of the run-off factor which are to be had, but because a careful study of them aids materially in training the judgment and in arriving at a clear and full conception of the problem.

In the present state of our knowledge, only sound judgment based upon experience and clear thinking, with a full conception of the various parts of the problem, can be relied upon for the selection of factors to use

in the design of storm-water conduits, because the existing gagings are lacking in the determination of important elements and the characteristics of districts are constantly changing with the growth of cities, so that a coefficient which might be applicable to-day will be totally insufficient for conditions likely to exist in the near future.

For an exact analysis of the relation between precipitation and run-off, it is necessary to know the true rainfall upon the district drained, including the distribution of rainfall over the entire area at all times during the storm, and the true storm run-off, including not only the quantity flowing past the gaging point at all times during and immediately before and after the storm, but the amount which could have been concentrated at this point if the conditions had been favorable. For instance, if the critical precipitation comes at the beginning of a storm when the flow in the sewers is small and the velocity of flow slight, a very considerable portion of the run-off from the surface will be required to fill the sewers. In this case the velocity of flow will be small, the time of concentration will be long, and the actual maximum rate of flow in the sewer will be materially less than the real rate of run-off. If, on the other hand, the critical precipitation occurs after a long period of moderately heavy rain, particularly if accompanied by melting snow, when the storage space in the sewers is largely filled and the velocity of flow is at a maximum, the quantity actually flowing in the sewer will represent very nearly the true run-off from the storm and the time of concentration will be a minimum. Whether or not the storm-water inlets are adequate to admit water into the sewer as rapidly as it reaches the inlets is also of importance.

Rainfall.—The true rate of precipitation upon the district gaged must be known for each instant during the storm. If the district is a small one, a single recording rain-gage near the center of the area may be sufficient; otherwise several such gages will be required, distributed over the area, since the intensity of rainfall frequently varies widely in comparatively short distances. It is extremely important that the gage clocks be carefully adjusted to keep correct time, and to agree with one another and with the sewer gages, otherwise the deductions drawn from the records of several gages, and also those relating to time of concentration as shown by a comparison between rain and sewer gages, may be materially in error.

The matter of travel of the storm is also of importance. It is evident that if downpour begins at the most distant point of the drainage area and travels toward the outlet, the resulting maximum flow in the sewer will have progressed some distance before the portions of the district nearest the outlet begin to contribute water. The result is a decreased time of concentration for such storms and an increased run-off as compared with a storm of uniform intensity over the entire district. Storms

in which the travel is in the reverse direction would have the opposite effect.

It is obvious that travel of storms can only be determined by a number of gages suitably located and with the clocks carefully regulated. So far as is known, no records which throw light upon this subject are to be had.

It is evident that where the rainfall record is that of a single gage, particularly if at a distance from the sewer district gaged, the inferences relating to time of concentration, area tributary at time of maximum discharge, and run-off factor, may be considerably in error.

Measurement of Run-off.—In the great majority of cases the estimation of flow has been accomplished by computing the quantity flowing in the sewer, by Kutter's formula, using an assumed value of the coefficient of roughness, and assuming the slope of water surface parallel to the invert of the sewer, a record of the depth of flow only being secured by means of an automatic gage. In many cases the resulting estimated run-off may be far from the truth. Horner has found (*Jour. West. Soc. Engs.*, Sept., 1913) that:

"There are marked differences between the grade of the sewer and the water surface grade. For example, in a 9-ft. sewer for one rain a depth of flow at one point of $4\frac{1}{2}$ ft. was observed; 1000 ft. downstream the depth was less than 4 ft., though several tributaries entered between, while 500 ft. further downstream the depth was over 5 ft......... The sewer is uniform as to grade, size and condition. The most reasonable explanation of these gagings is that the flow at the upper and lower gages is disturbed, in the case of the upper gage, by a curve 200 ft. upstream, and of the lower by a 3-ft. lateral discharging into the main sewer nearly at right angles 100 ft. above the gage."

The fact that storage in the sewers may result in a rate of flow much less than the rate of storm-water run-off has already been referred to.

Extent of Drainage Area Tributary.—In the case of a downpour of less duration than the time of concentration for the entire sewer district gaged, with the velocities obtaining at the moment of gaging, and including effect of the travel of the storm, it is evident that the corresponding run-off represents the discharge from an area less than the entire sewer district. Where this condition has been taken into consideration, as it has in some of the gagings, it appears that the area lying within the time-distance from the gaging point corresponding to the duration of the downpour has been assumed to be tributary. It is not evident whether or not this estimate has been based upon velocities of flow actually obtaining, but it seems more probable that at least in some cases maximum velocities have been assumed. It is evident, however, that even were the time-distances correctly computed, the maximum run-off from a downpour of, say, 8 minutes might come from the portion of the

area lying between, say, 10 and 18 minutes time-distance from the gaging point rather than between 0 and 8 minutes, particularly if the former is reinforced by the run-off resulting from a following rain at a lesser rate upon the district lying between 0 and 10 minutes time-distance. All determinations of run-off factor based upon less than the entire sewer district above the gaging point are therefore likely to be materially in error.

Characteristics of Sewer District Gaged.—Finally, bearing in mind possible inaccuracies in the determination of the coefficient of run-off, it is necessary to know accurately the characteristics of the district in order to form an opinion of the applicability of the coefficients to other districts. These characteristics are of three classes, permanent, semi-permanent, and temporary.

The principal characteristics which may be classed as permanent are the size and shape of the district, the surface slopes, and the character of the soil. Even these are not absolutely permanent, as they are all subject to alteration if extensive grading operations should be undertaken.

Semi-permanent characteristics, those which change but slowly, are the extent and kind of the impervious or nearly impervious surfaces, such as roofs and pavements, the extent to which the district is sewered, and the sizes and grades of the sewers. The last items are particularly important in their relation to velocity of flow and to storage in the sewers.

Temporary characteristics relate to conditions existing at the time of gaging, which may be modified radically within a period of at most a few hours. The most important are those relating to the condition of the ground and roofs, whether and to what extent they are wet or dry, frozen or covered with snow or ice. Other conditions of minor importance are temperature, wind, etc.

Inlet Time.—A matter of considerable importance in this connection, and one about which our definite information is very incomplete and unsatisfactory, is the "inlet time," or time required for the water falling upon the surface to reach the inlets or catch basins. It will be noted in the example worked out in the preceding chapter that various inlet times were assumed, depending upon the size, slope, and other characteristics of the several areas. In many cases, particularly for the smaller districts, the inlet time may constitute a large percentage of the total time of concentration. So important is this matter that the Sewer Department of St. Louis started a special investigation in 1913 to determine the time of inlet and the quantity of run-off for individual drainage districts.

"Two special cases have been taken for the work, one an area consisting principally of back yards and alleys in which the slope is slight; the other a whole block closely built and having steep grades. The inlets have been

reconstructed, and a chamber containing a V-notch weir built under the street between inlet and sewer. Bristol gages are installed to measure head on the weirs. All adjacent inlets have been enlarged and arranged so that no water can cross over from one inlet area to another, even in the heaviest rains; the exact extent and character of the inlet areas have been plotted. The values of the run-off factor from these small areas will be of great service in analyzing the results from gages in the sewers."—(Horner, *Jour. W. S. E.*, Sept., 1913, p. 703.)

Further comments on this subject may be found in Chapter VIII, on "Rational Methods".

Having called attention to the defects to which all the recorded storm-water gagings are subject in greater or less degree, it may be well to reiterate that notwithstanding their imperfections these gagings are still of great importance and should be studied carefully. It is much to be desired that the number of gagings should be greatly increased, particularly for small districts, and that the uncertainties and unsatisfactory conditions attending the earlier measurements be eliminated as far as possible. The work now (1914) in progress in several locations, especially in St. Louis, seems to offer promise of more extended and more precise information in the near future. The first step in the acquisition of complete and accurate data must always be the recognition and avoidance of all possible sources of error and uncertainty.

It is particularly to be borne in mind that in the determination of the run-off factor it is necessary in most cases to assume the time of concentration, and take the rate of precipitation corresponding to this time for comparison with the maximum rate of run-off. Material errors in the coefficient may result from erroneous estimation of the time of concentration, as is shown in Table 105 following, in which the results of storm-water gagings at Philadelphia are given. The coefficients resulting from assuming the time of concentration at 30, 40, 50 and 60 minutes are tabulated, and it will be seen that they vary widely.

The following tables contain the most important data relating to all gagings of storm-water flow in sewers, which have come to the attention of the authors. Additional data, apparently of great value, have appeared in the 1913 progress report of the Committee on Rainfall and Run-off of the Society of Municipal Engineers of the City of New York. These gagings are being continued, and no attempt has yet been made to interpret the results.

TABLE 92.—MEASUREMENTS OF STORM-WATER FLOW IN SEWERS IN BIRMINGHAM, ENGLAND

Measurements of D. E. Lloyd-Davies, reported in *Proc. I. C. E.*, vol. clxiv, p. 5. Rainfall from Edgbarton Observatory in Birmingham—Run-off computed from two automatic sewer gages for each sewer gaged.

Moseley Street Sewer.—Area drained 312.5 acres. Population 125 per acre. Area wholly impervious, 22 per cent. in street pavements, 78 per cent. roofs. Average slope of surface 1 in 60. Minimum time of concentration, 18 min.

Date 1904	Max. intensity of rainfall during time of concentration, in. per hr.	Max. resulting rate of run-off c.f.s. per acre	Coefficient = Ratio of run-off to rainfall
Jan. 10	0.330	0.304	0.92
Jan. 14	0.274	0.283	1.03
Jan. 26	0.122	0.101	0.83
Jan. 27	0.210	0.147	0.70
Jan. 30	0.280	0.262	0.94
Feb. 4	0.198	0.246	1.24
Feb. 8	0.297	0.329	1.11
Feb. 13	0.280	0.252	0.90
Mar. 8	0.100	0.088	0.88
Mar. 29	0.132	0.160	1.23
Apr. 14	0.165	0.124	0.75
Apr. 16	0.132	0.139	1.05
May 2	0.297	0.289	0.97
May 21	0.198	0.178	0.90
May 27	0.726	0.795	1.09
July 26	0.866	0.836	0.96
Aug. 5	0.462	0.193	0.42
Sept. 3	0.066	0.043	0.65
Sept. 12	0.326	0.110	0.34
Oct. 1	0.099	0.074	0.75
Nov. 7	0.099	0.065	0.66
Nov. 10	0.139	0.105	0.76
Nov. 21	0.229	0.127	0.55
Dec. 4	0.109	0.065	0.60
Dec. 5	0.264	0.091	0.34
Dec. 10	0.075	0.073	0.97
Dec. 12	0.145	0.110	0.76
Dec. 14	0.065	0.058	0.89

TABLE 92.—MEASUREMENTS OF STORM-WATER FLOW IN SEWERS IN BIRMINGHAM, ENGLAND (*Continued*)

Charlotte Road Sewer.—Area drained 232 acres. Impervious area 18 per cent., of which pavements constitute 10 per cent. Population 17 per acre. Minimum time of concentration, 12 min.

Date 1904	Max. intensity of rainfall during time of concentration, in. per hr.	Max. resulting rate of run-off c.f.s. per acre	Coefficient = Ratio of run-off to rainfall
Jan. 14	0.274	0.054	0.20
Jan. 26	0.122	0.025	0.20
Jan. 27	0.210	0.040	0.19
Jan. 30	0.280	0.074	0.26
Feb. 4	0.198	0.039	0.20
Feb. 8	0.297	0.081	0.27
Feb. 13	0.280	0.049	0.18
Mar. 8	0.100	0.029	0.29
Mar. 29	0.132	0.030	0.23
Apr. 14	0.165	0.032	0.20
Apr. 16	0.183	0.040	0.22
May 2	0.297	0.072	0.24
May 21	0.198	0.030	0.15
May 27	0.68	0.181	0.27
July 26	1.04	0.273	0.26
Aug. 5	0.675	0.128	0.19
Aug. 17	0.338	0.051	0.15
Aug. 22	0.165	0.048	0.29
Sept. 3	0.100	0.025	0.25
Sept. 12	0.420	0.098	0.23
Oct. 1	0.175	0.049	0.28
Bordesley Street Sewer.—Area drained 19.32 acres, 100 per cent. impervious surface. Population 146 per acre. Minimum time of concentration, 6.5 min.			
Jan. 27	0.360	0.346	0.96
Jan. 30	0.350	0.346	0.99
Feb. 13	0.354	0.346	0.98
Mar. 8	0.240	0.258	1.07
Mar. 29	0.240	0.304	1.26
Apr. 14	0.414	0.386	0.93
Apr. 16	0.368	0.386	1.05
May 1	0.644	0 760	1.18

TABLE 93.—GAGINGS OF STORM-WATER FLOW IN SEWERS IN CAMBRIDGE, MASS.

Reported by John R. Freeman, "Report on Charles River Dam," and interpreted by Samuel A. Greeley in *Jour. W. S. E.*, Sept., 1913.

	Shepard St.	Sherman St.
Tributary area, acres	56.5	68
Percentage roof area	12	10
Percentage street area	24	18
Percentage lawns and gardens	64	72
No. of houses	155	292
Houses from which roofwater runs directly to sewer	80	87
General slope of area	0.028	0.032
General character of soil	Sandy	Clay

Time of concentration not given; assumed by Greeley as 20 min. in both cases. Rain measured by ordinary gage between drainage areas, and by automatic gage about 1 mile distant.

Date, 1900	Time after beginning of rain, Hr. Min.	Av. rate of rainfall for 20 min., in. per hr.	Max. rate of discharge in sewer, c.f.s. per acre	Coefficient
Gagings during long steady rains				
Shepard St. Sewer				
Feb. 25	2–10	0.30	0.07	0.23
	3–55	0.34	0.13	0.38
	5–30	0.275	0.15	0.55
	6–20	0.27	0.16	0.59
May 3	5–25	0.43	0.05	0.14
	6–25	0.56	0.14	0.25
	8–25	0.30	0.095	0.32
Sherman St. Sewer				
May 3	5–35	0.43	0.028	0.65
	6–55	0.565	0.44	0.78
	8–35	0.33	0.33	1.00
Gagings of run-off from heavy summer showers				
Shepard St.				
July 25		1.00	0 32	0.32
Aug. 10		0.70	0.35	0.50
Aug. 27		1.80	0.51	0.28
Sherman St.				
July 25		1.00	0.45	0.45
Aug. 10		0.70	0.20	0.29
Aug. 27		1.80	0.70	0.39
Gagings of run-off from steady heavy rains, on ground previously saturated				
Shepard St.		0.365	0.17	0.47
		0.15	0.08	0.53
Sherman St.		0.365	0.365	1.00

Table 94.—Gagings of Storm-water Flow in Sewers in Cambridge, Mass.

Data from City Engr. Lewis M. Hastings to Run-off Committee of Boston Soc. C. E.

Date	Max. rate of rainfall, in. per hr.	Duration of this rate	Max. rate of run-off, c.f.s. per acre	Coefficient
Shepard Street Sewer. (See preceding table for description of district.)				
1900				
Feb. 12	0.28	1 hr. 10 m.	0.19	0.68[1]
Feb. 22	0.36	0–30	0.31	0.86[1]
Feb. 25	0.30	0–30	0.07	0.23[1]
Feb. 25	0.34	0–30	0.135	0.40
Feb. 25	0.27	0–35	0.15	0.56
Feb. 25	0.26	1–00	0.16	0.62
Feb. 25	0.20	0–40	0.09	0.45
May 3	0.56	1–0	0.13	0.23
May 3	0.30	0–50	0.10	0.33
Sept. 17	0.36	3–40	0.17	0.47
Sept. 18	0.90	0–30	0.135	0.15
Sept. 18	0.60	0–30	0.18	0.30
Sept. 21	0.62	0–25	0.22	0.35
Nov. 9	0.52	0–20	0.17	0.33
Nov. 25	0.15	0–30	0.08	0.53

[1] Ground frozen: no snow.

Bath Street Sewer.—Area 223 acres. Flow measured over weir. No data on impervious surface. Rains of less duration than 30 min. are omitted. Sandy soil. Flat slopes. Rain-gage about 1 mi. distant.

Nov. 29, '10	0.20	1–0	0.032	0.16
Feb. 4, '11	0.52	0–30	0.067	0.13
June 6, '11	0.23	1–0	0.045	0.20
Nov. 12, '11	0.35	1–0	0.05	0.14

Table 95.—Measurements of Storm-water Flow in Western Outfall Sewer, Louisville, Ky.

Data from J. B. F. Breed Chief Engr., Commissioners of Sewerage, to Run-off Committee, Boston Soc. of Civil Engineers.

Sewer 10.38 ft. diameter, slope 0.058% (from levels). draining 2500 acres. Depth of flow observed at three points a short distance apart; considerable difference was noted; the average used was 9.0 ft. and the discharge computed by Kutter's formula using $n = 0.015$. Rain-gage about 1000 ft. beyond boundary of district, and about 1¾ miles from center of district. A fully developed city district, about 36 per cent. impervious. Average surface slope about 0.004. Soil clayey. Computed time of flow in sewer from most distant point —73 minutes; time of concentration assumed as 80 minutes.

Date	Max. av. intensity of rainfall for 80 min., in. per hr.	Max. rate of run-off, c.f.s. per acre	Coefficient
June 9–10, '10	0 60	0.193	0.32

TABLE 96.—GAGINGS OF STORM-WATER FLOW IN SEWERS IN CAMBRIDGE, MASS.

Data from City Engr. Lewis M. Hastings to Run-off Committee of Boston Soc. of Civil Engineers.

Oxford Street Sewer.—Drainage area 400 acres, of which 16 per cent. is made up of pavements and 13 per cent. of roofs—total impervious area 29 per cent. Soil mostly gravelly, but about 10 per cent. of district is clayey. Slopes generally flat. Rain gage at city hall about 3/4 mile from gaging station and about 1 mile from center of district. Time of concentration computed as 40 minutes.

Date	Max. rate of rainfall, in. per hr.	Duration of this rate, hr. min.	Max. rate of run-off, c.f.s. per acre	Coefficient	Remarks
Apr. 23, 1909.......	0.13	1-0	0.016	0.12	Beginning of storm
Apr. 23, 1909... ..	0.18	0-50	0.037	0.21	2 hr. after beginning of rain
June 13-14, 1909....	0.58	1-0	0.08	0.14	Beginning of storm
June 13-14, 1909....	0.31	0-35	0.059	0.19	1½ hr. after beginning of rain
July 3	0.17	0-50	0.024	0.14	4½ hr. after beginning of rain
Sept. 1............	0.48	0-35	0.041	0.09	Beginning of storm
Sept. 1............	0.14	0-40	0.03	0.21	1½ hr. after beginning of rain
Sept. 27...........	0.25	1-15	0.044	0.18	Beginning of storm
Sept. 28...	0.08	1-0	0.015	0.19	½ hr. after beginning of rain
Sept. 28...........	0.14	0-35	0.022	0.16	5 hr. after beginning of rain
Sept. 28...........	0.15	1-20	0.031	0.21	10 hr. after beginning of rain
Sept. 28..	0.12	0-40	0 023	0.19	13½ hr. after beginning of rain

TABLE 97.—MEASUREMENTS OF STORM-WATER FLOW IN SEWERS IN MILWAUKEE, WIS.

Experiments of Logemann and Nommensen, reported in *Eng. News*, May 30, 1901, recomputed from published figures which contain error.

Gagings in 8-ft. sewer having slope of 0.0025. Rain determined by Weather Bureau automatic gage and checked by ordinary rain-gage; these gages 1 to 2 miles distant from sewer district but records considered applicable to storms reported. Total area of sewer district 1138 acres, of which 18.5 per cent. is occupied by streets. Fair average residential district, well built up. Several streets have block pavements or macadam, but most of them have gravel surface. Time of concentration for whole area with maximum velocity in sewer = 44 minutes; as computed for storms gaged, with velocities actually obtained, time of concentration ranged from 67 to 100 minutes. It was assumed that the proportion of the drainage area contributing to maximum flow was the same as the ratio between duration of rain at maximum rate and computed time of concentration under existing conditions. In only one storm was entire area tributary at time of maximum flow.

Date, 1898	Max. av. rate of rainfall observed, in. per hr.	Duration of this rainfall, min.	Time required for concentration with velocity actually obtained, min.	Corresponding percentage of total area contributing to max. flow	Precipitation on tributary area, c.f.s.	Max. rate of run-off, as gaged c.f.s.	Coefficient = ratio of run-off to precipitation
July 31	0.10	30	89	33 7	38.3	6.42	0.17
Aug. 2	0.28	85	75	100.0	318.0	65.1	0.20
Aug. 6	0.17	15	100	15.0	29.4	5.72	0.19
Aug. 23	0.72	25	67	37.0	303.0	117.0	0.38

TABLE 98.—MEASUREMENTS OF STORM-WATER FLOW IN SEWERS IN CHICAGO, ILL.

Gagings by Sanitary District of Chicago, reported by S. A. Greeley in *Jour. W. S. E.*, Sept., 1913. Weir measurement of discharge.

Winnetka; Cherry St. Sewer.—381 acres, residential. Population per acre = 4.5. Impervious area 10 per cent. District approximately rectangular, 1 × 1.6 mile. Rain-gage in S. W. corner of district, 3500 ft. from gaging point. Time of concentration 60 minutes. Limits of area tributary in 20, 30 and 40 minutes, were determined.

Date, 1912	Observed time of concentration, min.	Area tributary, acres	Max. av. rate of precipitation during time of concentration, in. per hr.	Corresponding precipitation on tributary area, c.f.s.	Max. flow gaged (excess over dry weather flow), c.f.s.	Coefficient	Remarks
July 20	75	381	0.16	62.1	13.5	0.22	Sudden moderate shower
July 20	50	320	0.20	64.0	15.0	0.23	12 hr. after previous storm
July 23	25	160	0.45	73.9	15.5	0.21	Short sharp shower
July 28	60	365	0.15	56.2	12.3	0.22	Second of two showers
Aug. 8	30	195	0.39	76.8	11.7	0.15	Short quick storm
Aug. 9	75	381	0.22	82.3	13.0	0.16	8 hours after previous storm
Nov. 12	20	130	0.90	116.8	20.2	0.17	Sharp short shower

Evanston; Davis St. Sewer.—Well built-up area of 420 acres, 20 per cent. impervious. Time of concentration 40 minutes.

May 28	45	420	0.32	134.0	30.0	0.22	
June 4	15	141	0.60	84.6	11.8	0.14	
July 13	10	100	2.10	210.0	27.0	0.13	
July 21	45	420	0.15	63.0	10.0	0.16	
Aug. 20	10	100	0.78	78.0	5.6	0 07	

Diversey Boulevard Sewer.—Area 725 acres, 22 per cent. impervious. Population 32.5 per acre. District 2.4 × 0.5 miles. Very flat; many of lots are lower than streets. Time of concentration for whole area 75 minutes; 580 acres tributary in 60 minutes. Nearest rain-gage is at Post office, 4.5 miles south: next at Evanston, 8.25 miles north. Intensities used have been obtained by proportioning between these two gages.

Oct. 11	....	....	0.32	....	....	0.23	Short storm, no previous rain.
Oct. 31	....	725	0.05	....	....	0.67	Long storm, ground soaked.

Robey Street Sewer.—Area 2513 acres; 5.6 miles long by 0.8 to 1.0 mile wide; 7.6 per cent. impervious. Population 15.5 per acre. Practically flat; most lots are below street level. Time of concentration for whole area 7 hours. Area tributary in storms of 2 to 4 hours, includes most of the impervious surface, amounting to about 15 per cent. of that area. Rain-gage about 1 mile distant.

Sept. 21	120	580	0.13	75.4	5.0	0.07	First part of storm
Sept. 28	120	580	0.07	40.6	6.0	0.15	Latter part of storm
Oct. 2	180	900	0.033	29.7	5.0	0.17	Last part of long storm
Oct. 8	120	580	0.06	34.8	3.5	0.10	First part of light rain
Oct. 22	195	990	0.11	110.0	18.0	0.16	Last shower in 6-hour storm
Oct. 30	210	1080	0.034	36.7	4.7	0.13	Long, light rain
Oct. 31	240	1290	0.052	67.0	18.0	0.27	Last shower of a 10-hour storm

Table 99.—Measurements of Storm-water Flow in Franklin Ave. Sewer, Hartford, Conn.

Results of computations by Metcalf and Eddy from data in paper by F. L. Ford in *Trans. Conn. Soc. C. E.*, 1900–01, p. 133: rain record is that of city hall gage, 1/2 mi. from center of sewer district. (Computations for storage in sewer apply only to trunk sewer; no allowance for storage in branch sewers.)

Gaged at South St. by recording float gage. Sewer 6 ft. diam., slope 0.002. Hydraulic grade assumed parallel to invert. Area drained 477 acres, residential, about one-half fairly thickly built up, remainder somewhat sparse. Density of population 12 per acre. Time of concentration estimated to exceed 25 minutes.

Date 1900	Condition of ground	Max. av. rate of rainfall for period = time of concentration, in. per hr.	Duration of precipitation at this rate, min.	Corresponding discharge at 100 per cent. run-off, c.f.s.	Observed time from beginning of max. rain to max. flow, min.	Rate of max. flow in sewer in excess of ord. flow, c.f.s.	Add for storage in sewer c.f.s.	Total storm-water run-off, c.f.s.	Coefficient; ratio of run-off to precipitation
July 25	Dry	0.75	40	360	40	40	24	64	0.18
July 25	Wet by previous rain	1.00	60	477	60	78	10	88	0.18
Oct. 8	Ditto	0.50	85	238	65	22	4	26	0.11

Gaged at Bond St., by recording float gage. Sewer 4 ft. diam., slope 0.003. Hydraulic grade assumed parallel to invert. Area drained 263.5 acres, residential, densely built up except for institution occupying about 50 acres. Density of population 15.5 per acre. Time of concentration estimated to exceed 22 minutes.

1901 Mar. 11	Covered with ice	0.35	120	167	?	195	23	218	1.30(?)
July 11	Dry	3.75	20	990	20	75	58	133	0.14
1902 Feb. 28	Wet by previous rain	1.00	20	263	?	60	12+	72	0.27

Table 100.—Measurements of Storm-water Flow in Sewer in Newton, Mass.

Data from Edwin H. Rogers, City Engineer, to Run-off Committee of Boston Society of Civil Engineers.

Hyde Brook drainage area, 350 acres, of which about 28 per cent. is impervious. Suburban residence district well developed. Small brook enclosed in covered masonry channel. Rain-gage within district, about halfway between center and gaging point. Computations by Kutter's formula using $n = 0.013$. Gaging point not very satisfactory, since grade changed 160 ft. above and 138 ft. below gaging point. Section of channel changed 193 ft. above and 292 ft. below gaging station; at this latter point the water fell over a flight of steps with a vertical fall of about 9 ft. Time of concentration, 20 minutes.

Date	Max. av. rate of rainfall for 20 min., in. per hr.	Max. rate of run-off c.f.s. per acre	Coefficient	Remarks
Sept. 4, 1907......	1.40	0.61	0.43	After 1¼ hr. of rain
Aug. 7, 1908......	2.55	0.71	0.28	Beginning of storm

TABLE 101.—MEASUREMENTS OF STORM-WATER FLOW IN SIXTH AVE. SEWER, MANHATTAN, NEW YORK

Gaged by Rudolph Hering in 1887–88 by recording float gage. (Data quoted from C. E. Gregory, *Trans. Am. Soc. C. E.*, vol. lviii, 1907, p. 464).

A well built-up and paved section in the lower part of the city, area 221 acres, regular surface slope of about 0.007. About 90 per cent. of area impervious, remainder grass. Population 170 per acre. Time of concentration 15 minutes not including time required for water to reach inlets, nearest recording rain-gage 2 miles distant.

Date	Max. av. rate of rainfall for period = time of of concentration, in. per hour.	Corresponding max. rate of run-off observed, c.f.s. per acre	Coefficient = ratio of run-off to precipitation	Remarks
Dec. 28, 1887	0.730	0.290	0.39	
Dec. 28, 1887	0.25	0.18	0.72	
Feb. 25, 1888	0.49	0.28	0.57	
Feb. 25, 1888	0.36	0.27	0.75	
June 26, 1888	2.367	1.022	0.43	
July 19, 1888	1.850	0.666	0.36	
Aug. 4, 1888	2.910	1.162	0.40	Near end of storm
Aug. 21, 1888	2 180	0.880	0.40	At beginning of storm
Aug. 21, 1888	1.347	0.470	0.34	Near end of storm
Aug. 21, 1888	1.20	0.65	0.54	
Aug. 21, 1888	1.07	0.90	0.84	

TABLE 102.—MEASUREMENT OF STORM-WATER FLOW IN NEW YORK AVE. SEWER IN WASHINGTON, D. C.

Reported by Capt. R. L. Hoxie in *Trans Am. Soc. C. E.*, vol. xxv, pp. 81–82.

Area drained, 436 acres, of which 200 acres are closely built, and streets paved with asphalt; 156 acres sparsely built, but with streets mostly paved with asphalt; 80 acres open park. (Probably 55 to 60 per cent. impervious). Time of concentration about 25 minutes.

Date	Max. rate of precipitation, in. per hr.	Duration, min.	Max. rate of run-off, c.f.s. per acre	Coefficient
June 28, '81.........	4.23	25	2.00 +	0.48 +
Area above gaging point 200 acres, nearly 100 per cent. impervious.				
1884..............	2.00	15	1.50 ±	0.75±
June 28, '85.........	0.90	37	0.90	1.00

TABLE 103.—MEASUREMENT OF STORM-WATER FLOW FROM SHIPLEY RUN DRAINAGE AREA, WILMINGTON, DEL.

Data from A. J. Taylor, Engineer of Sewers, to Run-off Committee of Boston Society of Civil Engineers.

Area, 174 acres, with 31 per cent. paved surfaces and 34 per cent roofs. Total impervious area = 65 per cent. Soil clayey, general surface slope about 4 per cent. Flow computed by Kutter's formula using $n = 0.015$. Gaging point not very satisfactory, as grade and section both changed 140 ft. above and 15 ft. below point where depth was gaged. Time of concentration not given; assumed not to exceed 20 minutes.

Date	Max. rainfall rate for 20 min., in. per hr.	Max. rate of run-off, c.f.s. per acre	Coefficient
July 25, 1908	3.90	3.1	0.79

Note.—Rain had been falling heavily, but at a somewhat lesser rate, for 35 minutes before the beginning of the downpour, which caused the maximum run-off.

TABLE 104.—MEASUREMENTS OF STORM-WATER IN FLOW IN NEWELL AVE. SEWER DISTRICT, PAWTUCKET, R. I.

From figures reported by George A. Carpenter, City Engineer, to Run-off Committee of Boston Society of Civil Engineers.

Drainage area 146 acres, of which 25 per cent. consists of pavements and 9 per cent. of roofs—total impervious area 34 per cent. Soil, sand and gravel covered with 2 ft. of loam. Average surface slope less than 2 per cent. Rain-gage about 1 mile distant. Slope of sewer changes at gaging point; no change in section for 2250 ft. above gage; outlet is 545 ft. below. Flow computed by Hazen-Williams formula with $c = 150$, using slope of sewer down-stream from gage. Time of flow in sewer from most distant point computed as 23¾ minutes. Concrete sewer with very smooth interior; $c = 150$ justified by careful check measurements.

Date	Max. rate of rainfall, in. per hr.	Duration of this rate, hr. min.	Max. rate of run-off, c.f.s. per acre	Coefficient	Time after beginning of storm
Spring months					
Mar. 25, '09	0.26	0–48	0.092	0.35	
Summer months					
Aug. 26, '08	0.50	0–30	0.16	0.32	
June 17, '10	0.22	0–50	0.071	0.33	
Aug. 15, '10	0.20	0–45	0.058	0.31	
Aug. 28, '10	0.24	0–35	0.064	0.27	
Fall months					
Oct. 20, '06	1.02	0–35	0.302	0.30	
Sept. 4–5, '07	0.69	0–25	0.299	0.43	
Sept. 28–29, '07	0.29	0–50	0.128	0.44	
Oct. 8, '07	0.46	0–50	0.189	0.43	2 hr.–40 m.
Nov. 4, '10	0.23	1–07	0.112	0.49	
Nov. 29, '10	0.17	1–40	0.044	0.26	4 hr.–20 m.
Winter months					
Dec. 23, '07	0.51	0–37	0.295	0.58	
Feb. 19, '08	0.70	0–54	0.462	0.66	3 hr.–15 m.
Feb. 26, '08	0.15	1–28	0.075	0.50	
Feb. 26, '08	0.22	1–0	0.192	0.87	
Dec. 7, '08	0.30	2–10	0.148	0.49	6 hr.
Feb. 10, '09	0.30	0–30	0.148	0.49	

TABLE 105.—MEASUREMENTS OF STORM-WATER FLOW IN SEWERS IN PHILADELPHIA, PA.

Gaging point in 13-ft. sewer at Twelfth and Diamond Sts. Data published in annual reports of Bureau of Surveys supplemented by information submitted to Run-off Committee of Boston Society of Civil Engineers by George S. Webster, Chief Engineer, Bureau of Surveys. Intensity of rainfall for periods from 10 to 60 minutes duration are given in the original reports and the ratio between run-off and the 30-, 50- and 60-minute precipitation rates has been computed from them.

Area drained 1360 acres, two-thirds of which is improved property. Time of flow in sewer at maximum velocity about 33 minutes. Time of concentration assumed as 40 minutes.

Date	Intensity of rainfall for 40 min. in. per hr.	Max. rate of flow in sewer, c.f.s. per acre	Coefficient	Ratio between max. rate of flow and rainfall rate for		
				30 min.	50 min.	60 min.
1903						
June 10......	1.64[1]	0.49	0.30	.....	.....	.....
June 10......	1.45	1.33	0.92	.....	.....	.....
1906						
May 28.......	1.39	1.02	0.73	0.62	0.90	1.04
June 16.......	1.71	1.05	0.61	0.47	0.70	0.83
Aug. 2........	1.23	0.88	0.72	0.58	0.77	0.92
Aug. 21......	1.32	0.94	0.71	0.60	0.86	1.03
Aug. 24......	0.77	0.71	0.92	0.76	1.04	1.17
Oct. 5........	1.94	1.02	0.53	0.45	0.63	0.70
1907						
May 16.......	0.89	0.87	0.98	0.79	1.02	1.10
July 18.......	1.65	0.94	0.57	0.46	0.71	0.85
July 20.......	1.25	0.89	0.71	0.56	.....	.....
July 20	....	0.55	....	0.59	.....	.....
Sept. 28......	0.77	0.53	0.69	0.54	0.83	0.93
1908						
May 22.......	1.42	0.84	0.59	0.45	0.71	0.85
June 15......	0.62	0.90	1.45	1.08	1.73	1.92
July 25	1.42	0.96	0.68	0.63	0.83	0.98
Aug. 7.......	....	0.70	....	0.64	.....	.....
Aug. 25......	1.00	0.72	0.72	0.65	0.85	1.02
Aug. 26......	0.83	0.70	0.84	0.80	1.06	1.17
1909						
Aug. 16.......	0.69	0.56	0.82	0.72	0.94	1.02
1910						
Aug. 8........	0.80	0.61	0.76	0.58	0.88	1.03
Aug. 19.......	1.30	0.94	0.72	0.60	0.80	0.90
Sept. 2.......	0.87	0.41	0.47	0.40	0.58	0.65
Sept. 6	1.45	1.06	0.73	0.56	0.91	1.10
Oct. 19–20....	0.85	0.59	0.70	0.68	0.71	0.75
1911						
June 13.......	....	1.03	....	0.67	.....	.....
July 17.......	1.35	1.09	0.81	0.64	0.96	1.16
Aug. 15......	0.75	0.88	1.18	0.88	1.47	1.60
Aug. 30.......	0.92	0.88	0.95	0.77	1.09	1.20
Sept. 11......	0.59	0.46	0.78	0.69	0.98	0.98
Nov. 6.......	0.60	0.51	0.85	0.81	0.89	0.92
1912						
Feb. 26.......	1.24	1.01	0.82	0.62	1.01	1.19
Mar. 12......	1.00	0.81	0.81	0.74	0.90	1.00
Mar. 15......	1.24	0.98	0.79	0.78	0.92	1.03
Nov. 1.......	0.45	0.59	1.31	0.95	1.79	1.79
Nov. 7.......	....	1.06	....	0.77	.....	.....

[1] 45 minutes.

TABLE 106.—MEASUREMENTS OF STORM-WATER FLOW IN SEWERS IN ROCHESTER, N. Y.

Gagings of Emil Kuichling, data from *Trans. Am. Soc. C. E.*, vol. xx, 1889, p. 1. Gagings made by max. flow gages, determining slope from records of pairs of gages. Rain carefully measured but not by automatic gages.

District I.—Area 356.9 acres, residential; about half has population of 35 per acre; remainder sparsely settled, agricultural. Soil mostly clayey loam. Earth roads. Max. time of flow in sewers estimated at 34 minutes. Time of concentration 44 minutes. Impervious area 15 per cent.

Date	Max. intensity of rainfall, in. per hr.	Corresponding precipitation on drainage area, c.f.s.	Max. sewer discharge, c.f.s.	Coefficient = ratio of discharge to precipitation	Remarks
Dec. 10, '87.	0.31	110.6	15.3	0.14	Preceded and followed by lighter rains
Apr. 5, '88..	0.24	85 7	8.94	0.10	Preceded and followed by lighter rains
May 4.....	0.30	107.1	7.32	0.07	Preceded and followed by lighter rains
May 9.....	1.32	469.5	77.0	0.16	Sudden shower, followed by light rain
May 12....	0.30	107.1	11.8	0.11	Preceded and followed by lighter rain
May 26....	1.00	356.9	30.8	0.09	Preceded and followed by lighter rain
June 2.....	0.40	142.8	7.81	0.06	Sudden shower " " " "
June 24....	1.55	553.2	40.7	0.07	Sudden shower " " " "
June 24....	2.62	935.1	58.8	0.06	Sudden shower " " " "
June 28....	0.80	285.5	40.7	0.14	Preceded and followed by lighter rain
July 11.....	0.76	271.2	19.9	0.07	Heavy shower preceded by lighter rain
July 18.....	0.75	267.7	20.5	0.08	Preceded and followed by lighter rain
Aug. 4.....	1.00	356.9	16.5	0.05	Sudden shower " " " "
Aug. 16....	1.62	576.8	27.2	0.05	Sudden shower " " " "
Aug. 17....	1.33	475.9	25.8	0.06	Sudden shower " " " "
Aug. 26....	2.50	892.4	35.3	0.04	Intensity estimated roughly
Sept. 16....	0.47	167.7	33.3	0.20	Sudden shower followed by lighter rain

District IV.—Well-developed area of 128.7 acres; about 4800 ft. long and 1200 ft. wide. Av. density of population 32 per acre. Many business blocks in one portion, remainder residential. Soil mostly clayey loam. About ⅓ of streets paved, mostly with macadam, but some stone block and asphalt. Time of flow in sewer, 18 minutes. Time of concentration, 26 minutes. Impervious area about 30 per cent.

Dec. 10, '87.	0.31	39.9	9.27	0.24	Preceded and followed by lighter rain
Apr. 5, '88..	0.24	30.9	4.80	0.16	Preceded and followed by lighter rain
May 4.....	0.30	38.6	5.56	0.14	Preceded and followed by lighter rain
May 9.....	1.00	128.7	33.7	0.26	Intensity estimated
May 12....	0.30	38.6	6.09	0.16	Preceded and followed by lighter rain
May 26....	1.00	128.7	33.3	0.26	Preceded and followed by lighter rain
June 2.....	0.40	51.5	4.67	0.09	Sudden shower " " " "
June 24....	2.62	337.2	71.3	0.21	Sudden shower " " " "
June 28....	0.80	103.0	29.5	0.29	Preceded and followed by lighter rain
July 11.....	0.76	97.8	15.4	0.16	Heavy shower preceded by lighter rain
July 18.....	0.75	96.5	11.8	0.12	Preceded and followed by lighter rain
Aug. 4.....	1.00	128.7	12.8	0.10	Sudden shower " " " "
Aug. 16....	1.62	208.0	25.9	0.13	Sudden shower " " " "
Aug. 17....	1.33	171.6	14.9	0.09	Sudden shower " " " "
Aug. 26....	2.50	321.8	39.3	0.12	Sudden shower " " " "
Sept. 16....	0.47	60.5	23.1	0.38	Sudden shower " " " "

District IX.—Well-developed residential district of 133 acres; population 36 per acre. Dwellings mostly large and rather close together. Streets mostly macadam or gravel. Soil generally loamy. Time of flow in sewer, 15 minutes. Time of concentration, 23 minutes. Impervious area about 38 per cent.

Dec. 10, '87.	0.31	41.2	17.1	0.42	Preceded and followed by lighter rain
Apr. 5, '88..	0.24	31.9	12.2	0.38	Preceded and followed by lighter rain
May 4.....	0.30	39.9	14.4	0.36	Preceded and followed by lighter rain
May 9.....	0.75(?)	99.8	29.0	0.29	Intensity estimated
May 12....	0.30	39.9	11.8	0.30	Preceded and followed by lighter rain
May 26....	1.00	133.0	24.9	0.19	Preceded and followed by lighter rain
June 2.....	0.40	53.2	20.0	0.38	Sudden shower " " " "
June 24....	2.62	348.5	46.0	0.13	Sewer surcharged
June 28....	0.80	106.4	37.5	0.35	Preceded and followed by lighter rain
July 11.....	0.76	101.1	22.1	0.22	Heavy shower preceded by lighter rain
July 18.....	0.75	99.8	14 8	0.15	Preceded and followed by lighter rain
Aug. 4.....	1.00	133.0	19.9	0.15	Sudden shower " " " "
Aug. 16....	1.62	215.0	38.2	0.18	Sudden shower " " " "
Aug. 17....	1.33	177.3	21.1	0.12	Sudden shower " " " "
Aug. 26....	2 50	332.5	46.0	0.14	Sewer surcharged

District X.—Well-developed area of 25.1 acres, with population of 40 per acre, long narrow strip containing many business blocks and apartment houses, as well as single residences. Streets mostly macadamized. Soil clayey loam. Time of flow in sewers, 10 minutes. Time of concentration, 16 minutes. Impervious area about 50 per cent.

Dec. 10, '87.	0.31	7.80	4.54	0.58	Preceded and followed by lighter rain
May 4, '88..	0.30	7.54	4.89	0.65	Preceded and followed by lighter rain
May 9.....	0.75 ?	18.8	9.81	0.52	Intensity estimated
May 12....	0 30	7.54	2.66	0.35	Preceded and followed by lighter rain
May 26....	1.00	25.1	7.94	0.32	Preceded and followed by lighter rain
June 24....	2.62	65.8	21.0	0.32	Sewer surcharged. Flow estimated at max. capacity before surcharging
June 28....	0.80	20.1	7.09	0.35	Preceded and followed by lighter rain
July 11.....	0.76	19.1	8.01	0.41	Heavy shower preceded by lighter rain
July 18.....	0.75	18.8	4.70	0.25	Preceded and followed by lighter rain
Aug. 16....	1.62	40.6	10.0	0.25	Sudden shower. " " " "
Aug. 17....	1.33	33.5	6.17	0.18	Sudden shower " " " "
Aug. 26....	2.50	62.8	21.0	0.34	Sewer surcharged

District XVII.—Well-developed area of 92.3 acres. Population 35 per acre. One-fifth of streets paved with asphalt, one-fourth with stone block, remainder macadam and gravel. Buildings mostly single residences; some business blocks and apartments. Soil clayey loam. Half of the area is nearly level. Max. time of flow in sewers, 16 minutes; time of concentration, 24 minutes. Impervious area about 30 per cent.

Dec. 10, '87.	0.31	28.6	7.43	0.26	Preceded and followed by lighter rain
Apr. 5, '88..	0.24	22.2	4.61	0.21	Preceded and followed by lighter rain
May 4.....	0.30	27.7	7.82	0.28	Preceded and followed by lighter rain
May 9.....	0.75 ?	69.2	18.0	0.26	Intensity estimated
May 12....	0.30	27.7	4 70	0.17	Preceded and followed by lighter rain
May 26....	1.00	99.3	10.8	0.12	Preceded and followed by lighter rain
June 2.....	0.40	36.9	3.23	0.09	Sudden shower
June 24....	2.62	241.8	28.5	0.12	Sewer surcharged
June 28....	0.80	73.8	27.7	0.37	Preceded and followed by lighter rain
July 11.....	0.76	70.1	13.6	0.19	Heavy shower preceded by lighter rain
July 18.....	0.75	69.2	7.14	0.10	Preceded and followed by lighter rain
Aug. 4.....	1.00	92.3	12.7	0.14	Sudden shower " " " "
Aug. 16....	1.62	149.2	28.5	0.19	Sewer surcharged
Aug. 17....	1.33	123.1	10.9	0.09	Sudden shower " " " "
Aug. 26....	2.50	230.8	28.5	0.12	Sewer surcharged
Sept. 16 ..	0.47	43.4	16.1	0.37	Sudden shower " " " "

CHAPTER X

SEWER PIPE

Until recently, sewer pipe was given thicknesses which were the net result of the experience of makers and users, theory having little part in settling such dimensions. Recently, however, the great increase in the use of vitrified clay and cement pipe for sewers and drains, and the steady complaint of breakage with both classes, have led to both theoretical and experimental researches into the subject. It is naturally divided into two parts, the pressures which sewer pipe must resist and the stresses which are produced in the shell of a pipe.

INTERNAL PRESSURE UPON PIPE

The stress due to the internal pressure upon pipe is indicated by the formula

$s = \frac{pr}{t}$, $t = \frac{pr}{s}$, in which

s = tension in pounds per square inch upon the pipe,
p = pounds pressure per square inch of water in the pipe,
r = radius of the pipe in inches,
t = thickness of the pipe in inches.

In general it may be said that the working stress should not exceed from one-fourth to one-fifth of the ultimate strength of the material, if reasonably ductile as steel. In such brittle material as cast iron a much larger factor of safety than four or five is used, as appears in the following paragraph.

PRESSURE IN TRENCHES

One of the earliest attempts to ascertain the pressures produced in a trench by backfilling, was made by August Frühling in "Die Entwässerung der Städte," one of the volumes of Franzius and Sonne's "Handbuch der Ingenieur-Wissenschaften." He assumed that the vertical pressure due to backfilling, increased at a diminishing rate as the depth increased, until at a depth of 5 m. no further increase occurred. Further, he assumed that the total pressure at any depth varied according to a parabolic law. From these assumptions he deduced the following formula:

$$P = w\left(\frac{A}{3} - \frac{(A-t)^3}{3A^2}\right)$$

where P is the pressure per square meter of horizontal surface, w is the weight of a cubic meter of the backfill and A is the depth below which there is no increase in P. If this expression is transformed to English measures and w is taken at 100 lb. per cubic foot, the formula becomes:

$$p = 100t - 6.07t^{20.124} + t^3$$

where p = pressure in pounds per square foot at a depth of t ft.

Barbour Experiments.—The Frühling formula has been rarely if ever used in the United States, where until 1910–11 experiments by F. A. Barbour (*Jour. Assoc. Eng. Soc.*, Dec., 1897) were the basis of most discussions of the subject. His tests were made by placing a modified hydraulic ram in the bottom of a 13-ft. trench and supporting a platform on the plunger. Sheeting was placed across the trench at each end of the platform, so as to confine the backfill placed on the latter. This series of experiments was not utilized in developing a formula, but the results were expressed in a number of curves. These give smaller pressures than the Frühling formula at depths less than about 15 ft. and greater pressures below 15 ft.

Hazen's Analysis.—Basing his calculations upon Prof. Talbot's work on the strength of thin rings under external pressure, Allen Hazen suggested tentatively (*Jour. N. E. W. W. Assoc.*, May, 1911), two formulas for determining how thick a pipe must be to carry the stresses due to the backfilling as computed by Talbot's formula, and at the same time to carry a given internal pressure with 50 per cent. increase for water ram. An abstract of his statement follows:

Let d = diameter in inches,
t = thickness in inches,
F = depth of backfill above top of pipe in feet.
s = permissible stress in pounds per square inch in cast-iron pipe, which I now take as 4400 lb. for an ultimate tensile strength of 22,000 lb., with a factor of safety of 5.
W = weight of fill over 1 lin. in. of pipe at the rate of 115 lb. per cubic foot, the outside diameter of pipe being taken as 5 per cent. greater than d.

$$W = Fd\frac{1.05\times 115}{144} = 0.84Fd$$

M = breaking moment normally present from backfill, according to Talbot = 1/16 WD, D being the average diameter of the shell, which is about $1.025d$,

$$M = \frac{1}{16}\,1.025d(0.84Fd) = 0.0538Fd^2$$

Resulting maximum circumferential stress in metal in pounds per square inch, obtained by applying the usual formula,

$$M = \frac{1}{6}sbt^2, \ b \text{ in this case being } 1.$$

$$s_1 = \frac{6M}{t^2} = \frac{0.323Fd^2}{t^2}$$

The stress available for resisting the water pressure is 4400 minus this amount. Of this, one-third is allowed for water ram and two-thirds for static pressure.

The stress allowable for resisting the static pressure is thus

$$s_2 = \frac{2}{3}\left(4400 - 0.322\frac{Fd^2}{t^2}\right)$$

H = head in feet that can be carried by a given stress;

$$s = \frac{r}{t} \text{ lb. pressure per square inch}$$

$$= \frac{d}{2t} \times \frac{H}{2.31} \text{ and}$$

$$H = s\left(4.62\frac{t}{d}\right);$$

and for s_2 as reached above,

$$H = 3.08\frac{t}{d}\left(4400 - 0.322\frac{Fd^2}{t^2}\right) = 13{,}500\frac{t}{d} - 0.99F\frac{d}{t}$$

If we had taken the weight of the earth backfill as 116 lb. per cubic foot, the 0.99 would have been unity, and we may make it unity for the purpose of simplifying the formula. We shall then have

$$H = 13{,}500\frac{t}{d} - F\frac{d}{t}$$

Our specifications allow a variation in the thickness of casting of 0.10 in. for large pipe. To insure that the stress shall not exceed the calculated amount at any point, if we could be sure that the specifications were literally complied with, it would only be necessary to add 0.10 in. to the computed thickness. This rule might be adopted for country work and where an occasional break in the pipe would not be of the greatest importance. For city work, or where a break might do great damage, it would seem better to add 0.25 in. to the computed thickness, this being the allowance made in the Brackett formula in all cases for this purpose.

Solving the last equation for t, and making this addition, we have

For country work:

$$t = 0.10 + \frac{d}{27{,}000}(H + \sqrt{54{,}000F + H^2})$$

For city work:

$$t = 0.25 + \frac{d}{27{,}000}(H + \sqrt{54{,}000F + H^2})$$

This formula is not suggested as in any way final, but only for the purpose of discussion, and with the idea that it may possibly have in it some elements of a more rational calculation than are contained in the old formulas.

These formulas certainly lead to conservative results, as considerably lighter weight pipe than that indicated by them as necessary has been successfully used in different places. Thus Leonard Metcalf reported during the discussion cases in which he had successfully used 20- and 24-in. pipe of the New England Water Works Association Class A standard, at depths of 18 ft. more or less, though under but slight internal pressure.

Iowa Investigations by Marston.—The results of an elaborate investigation of the subject, lasting several years, were made public in Bulletin 31 of the Engineering Experiment Station of the Iowa State College of Agriculture. This was written by Prof. Anson Marston, director of the station, and A. C. Anderson, and contains the first well-developed comprehensive theory of the subject which was also checked by numerous experiments.

These authors use in their analytical treatment of pressures in trenches practically the same method that was developed by Janssen for the pressures in grain bins (Ketchum's "Retaining Walls, Bins and Grain Elevators"). This gives for the weight on the pipe $W = CwB^2$, in which W is the total weight per unit length of pipe, C is a coefficient in which allowance is made for the ratio of the width and depth of the trench and for the coefficient of friction of the backfill against the sides of the trench; w is the weight of a unit volume of the backfill, and B is the width of the trench a little below the top of the pipe. The values of C are given in Table 107.

The approximate averages of a large number of measurements of weights and frictional properties of different classes of backfilling are given in Table 108. Within the range of ordinary ditch-filling materials, it takes a large difference in the values of the friction coefficients to make a material difference in the weight carried by the pipe. Marston and Anderson point out that the real difficulty in selecting the proper values from the table lies in deciding upon safe and reasonable allowances for the probable saturation of the materials under actual ditch conditions.

The approximate maximum loads on pipes in trenches of different widths and depths are given in Table 109. The investigations of Marston and Anderson have convinced them that a 12-in. pipe will have to carry the same load as an 18-in. pipe, if each is placed in the bottom of a 24-in. trench, other things being similar. When a wide trench is necessary for construction reasons, they believe that, in firm soil, the load can be greatly diminished by stopping the wide trench a few inches above the top of the pipe and then excavating the narrowest trench in which it is practicable to lay the pipe, making special enlargements for the bells, if necessary.

Their experiments to test the accuracy of the theory upon which this

and their other tables were based were made by weighing the load on different lengths of pipes hung at different depths in trenches, from a system of levers ultimately ending on the platform of scales. Particular care was taken to avoid all test conditions likely to cause uncertainty regarding the accuracy of the results, and where doubt arose the tests were repeated, with or without modification, until uncertainty was eliminated.

In commenting on Table 109, Marston and Anderson point out that the side pressure of the filling materials against the sides of the trench develops a frictional resistance which helps to carry part of the weight. This frictional resistance relieves part of the vertical pressure near the sides of the trench, so that at the level of the top of the pipe the vertical pressure of the filling materials, they state, is much greater in the middle of the trench than at the sides. Moreover, there is some arching effect on each side about 45-deg. down from the top of the pipe, and the comparatively horizontal top of the pipe is more solid and unyielding than the side filling material. Hence the trench fill above the pipe receives only a negligible support in ditches of ordinary width from the fill at the sides. For an extremely wide trench in proportion to the diameter, this principle would no longer hold. Imperfections in the side filling and tamping probably decrease the applicability of the principle.

Most analytical discussion of the pressures in trenches has been based upon the assumption of vertical sides. In many cases the sides of the trench widen outward from its bottom, a condition which was investigated both analytically and experimentally by Marston and Anderson. An arching action apparently takes place, they found, between the sides of the trench and points at the ends of the top quadrant of the pipe. Above the elevation of these 45-deg. points, the material along the sides settles less than that in the center of the trench. The investigations referred to led to the conclusion that in these wedge-shaped trenches the proper width to substitute for B in the formula $W = CwB^2$ and to use as the width of the trench in Table 109, is the width at the height of the 45-deg. points on the pipe circumference, just a little below the top of the pipe.

The pressure of the backfilling is not the only load which may come on the pipe, for the fresh fill may be called upon to support a heavy road roller or the wheels of a truck, and under some circumstances piles of paving materials, lumber or brick may be put directly on top of the backfilling for a considerable distance along its axis.

In order to determine the effect of such long excess loads Marston and Anderson carried on an analytical and experimental investigation. They found that if this extra load, per unit of length of trench, is regarded as unity, the decimal part of it which is transmitted to the pipe

These formulas certainly lead to conservative results, as considerably lighter weight pipe than that indicated by them as necessary has been successfully used in different places. Thus Leonard Metcalf reported during the discussion cases in which he had successfully used 20- and 24-in. pipe of the New England Water Works Association Class A standard, at depths of 18 ft. more or less, though under but slight internal pressure.

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Their experiments to test the accuracy of the theory upon which this

TABLE 108.—PROPERTIES OF DITCH-FILLING MATERIALS (MARSTON AND ANDERSON)

Material	Weight of filling, lb. per cu. ft.	Ratio of lateral to vertical earth pressures	Coefficient of friction againt sides of trench	Coefficient of internal friction
Partly compacted damp top soil	90	0.33	0.50	0.53
Saturated top soil	110	0.37	0.40	0.47
Partly compacted damp yellow clay	100	0.33	0.40	0.52
Saturated yellow clay	130	0.37	0.30	0.47
Dry sand	100	0.33	0.50	0.55
Wet sand	120	0.33	0.50	0.57

An example of the possible use of the table is given by Marston and Anderson in a discussion of the probable correctness of the general impression that more damage is done to pipe with a small depth of cover than to those in deep trenches, and that more damage is done during tamping than is frequently considered probable by those who draw specifications for pipe. The maximum pressure P on the backfill, resulting from the shock of a blow of a rammer, is $2TF/f$, where T is the weight in pounds of the rammer, F is the height in feet of the fall of the rammer, and f is the compression of the backfill under one blow of the rammer at the end of the tamping.

The data for the example of the use of the formula may be taken from a discussion by J. N. Hazlehurst (*Jour. Assn. Eng. Socs.*, vol. xix). Here the original "very thorough" tamping was done with a 40-lb. rammer on a 6-in. clay cover, resulting in some cracking, while later the use of a 30-lb. rammer on a 12-in. fill had no such result. If it is assumed that very thorough tamping on a 6-in. cover is such as would produce a final compression f of 0.01 ft. under one blow, and the height of fall F was 0.5 ft., then with a 40-lb. rammer $P = 4000$ lb. If the rammer had a face width of 0.67 ft., then the ratio of depth of cover to the width over which the load was applied $H/b = 0.5/0.67$ was 0.75. The percentage of P reaching the pipe would be, from Table III, about 71. Hence about 2800 lb. would be directly transmitted to an 8×8-in. area of pipe. With the lighter rammer, f would probably be a little larger, say 0.015 ft., because the cover was 1 ft. instead of 0.5 ft. The same method of computation as in the first case shows that the pressure on the 8×8-in. area would be about 1000 lb. The correctness of the opinion occasionally expressed regarding the use of a rather thick cover and light rammer in the lower part of the trench is confirmed by this analytical method of investigation.

If sheeting is left in the trench, but the rangers are removed, the friction between the backfill and the sides of the trench is manifestly

decreased and the load on the pipe increased. The Marston and Anderson experiments indicate that this increase is from 8 to 15 per cent. and the experiments by F. A. Barbour (*Jour. Assn. Eng. Socs.*, 1897) confirm this conclusion. If the rangers are left in place, the load coming on the pipes would probably be about the same as in unsheeted trenches, according to both theory and experiment by Barbour.

TABLE 109.—APPROXIMATE MAXIMUM LOADS, IN POUNDS PER LINEAR FOOT, ON PIPE IN TRENCHES, IMPOSED BY COMMON FILLING MATERIALS. (MARSTON AND ANDERSON).

Depth of fill above pipe	Breadth of ditch at top of pipe									
	1 ft.	2 ft.	3 ft.	4 ft.	5 ft.	1 ft.	2 ft.	3 ft.	4 ft.	5 ft.
	Partly compacted damp top soil; 90 lb. per cubic foot					Saturated top soil; 110 lb. per cubic foot				
2 ft.	130	310	490	670	830	170	380	600	820	1,020
4 ft.	200	530	880	1,230	1,580	260	670	1,090	1,510	1,950
6 ft.	230	690	1,190	1,700	2,230	310	870	1,500	2,140	2,780
8 ft.	250	800	1,430	2,120	2,790	340	1,030	1,830	2,660	3,510
10 ft.	260	880	1,640	2,450	3,290	350	1,150	2,100	3,120	4,150
	Dry sand; 100 lb. per cubic foot					[1]Saturated sand; 120 lb. per cubic foot				
2 ft.	150	340	550	740	930	180	410	650	890	1,110
4 ft.	220	590	970	1,360	1,750	270	710	1,170	1,640	2,100
6 ft.	260	760	1,320	1,890	2,480	310	910	1,590	2,270	2,970
8 ft.	280	890	1,590	2,350	3,100	340	1,070	1,910	2,820	3,720
10 ft.	290	980	1,820	2,720	3,650	350	1,180	2,180	3,260	4,380
12 ft.	300	1,040	2,000	3,050	4,150	360	1,250	2,400	3,650	4,980
14 ft.	300	1,090	2,140	3,320	4,580	360	1,310	2,570	3,990	5,490
16 ft.	300	1,130	2,260	3,550	4,950	360	1,350	2,710	4,260	5,940
18 ft.	300	1,150	2,350	3,740	5,280	360	1,380	2,820	4,490	6,330
20 ft.	300	1,170	2,420	3,920	5,550	360	1,400	2,910	4,700	6,660
22 ft.	300	1,180	2,480	4,060	5,800	360	1,420	2,980	4,880	6,960
24 ft.	300	1,190	2,540	4,180	6,030	360	1,430	3,050	5,010	7,230
26 ft.	300	1,200	2,570	4,290	6,210	360	1,440	3,090	5,150	7,460
28 ft.	300	1,200	2,600	4,370	6,390	360	1,440	3,120	5,240	7,670
30 ft.	300	1,200	2,630	4,450	6,530	360	1,440	3,150	5,340	7,830
	Partly compacted damp yellow clay; 100 lb. per cubic foot					[1]Saturated yellow clay; 130 lb. per cubic foot				
2 ft.	160	350	550	750	930	210	470	730	1,000	1,240
4 ft.	250	620	1,010	1,400	1,800	340	840	1,330	1,870	2,370
6 ft.	300	830	1,400	1,990	2,580	430	1,140	1,900	2,630	3,410
8 ft.	330	990	1,720	2,500	3,250	490	1,380	2,360	3,360	4,400
10 ft.	350	1,110	2,000	2,920	3,880	520	1,570	2,760	3,980	5,270
12 ft.	360	1,200	2,220	3,320	4,450	540	1,730	3,100	4,560	6,050
14 ft.	370	1,280	2,410	3,650	4,950	560	1,850	3,410	5,050	6,760
16 ft.	370	1,330	2,570	3,950	5,400	570	1,940	3,660	5,510	7,440
18 ft.	380	1,380	2,710	4,210	5,810	570	2,020	3,880	5,930	8,060
20 ft.	380	1,410	2,830	4,450	6,180	580	2,090	4,070	6,280	8,610
22 ft.	380	1,430	2,920	4,640	6,500	580	2,140	4,240	6,610	9,130
24 ft.	380	1,450	3,000	4,820	6,800	580	2,180	4,380	6,910	9,590
26 ft.	380	1,470	3,060	4,980	7,080	580	2,210	4,500	7,160	10,010
28 ft.	380	1,480	3,120	5,100	7,310	580	2,240	4,610	7,380	10,430
30 ft.	380	1,490	3,170	5,230	7,530	580	2,260	4,700	7,590	10,780

[1] These two sub-tables contain the most important figures for practical use.

TABLE 110.—PROPORTION OF LONG SUPERFICIAL LOADS ON BACKFILLING WHICH REACHES THE PIPE IN TRENCHES WITH DIFFERENT RATIOS OF DEPTH TO WIDTH AT TOP OF PIPE (MARSTON AND ANDERSON)

Ratio of depth to width	Sand and damp top soil	Saturated top soil	Damp yellow clay	Saturated yellow clay
0.0	1.00	1.00	1.00	1.00
0.5	0.85	0.86	0.88	0.89
1.0	0.72	0.75	0.77	0.80
1.5	0.61	0.64	0.67	0.72
2.0	0.52	0.55	0.59	0.64
2.5	0.44	0.48	0.52	0.57
3.0	0.37	0.41	0.45	0.51
4.0	0.27	0.31	0.35	0.41
5.0	0.19	0.23	0.27	0.33
6.0	0.14	0.17	0.20	0.26
8.0	0.07	0.09	0.12	0.17
10.0	0.04	0.05	0.07	0.11

Note.—Curves based on this table are given in Fig. 181.

TABLE 111.—PROPORTION OF SHORT SUPERFICIAL LOADS ON BACKFILLING WHICH REACHES THE PIPE IN TRENCHES WITH DIFFERENT RATIOS OF DEPTH TO WIDTH (MARSTON AND ANDERSON)

Ratio of depth to width	Sand and damp top soil		Saturated Top soil		Damp yellow clay		Saturated yellow clay	
	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.
0.0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
0.5	0.77	0.12	0.78	0.13	0.79	0.13	0.81	0.13
1.0	0.59	0.02	0.61	0.02	0.63	0.02	0.66	0.02
1.5	0.46	....	0.48	....	0.51	....	0.54	....
2.0	0.35	....	0.38	....	0.40	....	0.44	....
2.5	0.27	....	0.29	....	0.32	....	0.35	....
3.0	0.21	....	0.23	....	0.25	....	0.29	....
4.0	0.12	....	0.14	....	0.16	....	0.19	....
5.0	0.07	....	0.09	....	0.10	....	0.13	....
6.0	0.04	....	0.05	....	0.06	....	0.08	....
8.0	0.02	....	0.02	....	0.03	....	0.04	....
10.0	0.01	....	0.01	....	0.01	....	0.02	....

Note.—Curves based on this table are given in Fig. 181.

The experimental and theoretical work which has been referred to at length relates to new backfilling. There is abundant evidence that in many cases the maximum load on the sewer pipe is not imposed until the first heavy rain saturates the trench and possibly puts the pipe under internal pressure. Reports from engineers in Iowa confirm statements made at an earlier date by engineers with experience in eastern states, that lines of sewer pipe sometimes crack long after

they were put in place. Such evidence indicates, therefore, that in considering the necessary strength of pipe the values of the trench pressures determined by Marston and Anderson may probably be accepted as reasonably accurate, particularly in view of their careful investigation of lines of drains and sewers in Iowa which have not failed, the average factor of safety in these cases being about 1.65, estimated from the average trench pressures and pipe strengths determined by the investigation.

STRENGTH OF PIPE

The theoretical analysis of the strength of pipe under external loads is that of thin elastic rings, and takes various forms under different assumptions regarding the loading. An explanation of it is given by Prof. A. N. Talbot, in Bulletin 22 of the Engineering Experiment Station of the University of Illinois, in which he reports tests of cast-iron and reinforced concrete culvert pipe. Marston and Anderson have given the results of an analysis in which the weight of the pipe, as well as that of the backfilling, is taken into consideration. For all practical purposes, three assumed conditions of loading will be sufficient to guide the engineer, viz., concentrated loads at the top and bottom of the vertical diameter of the pipe, uniformly distributed vertical loads above and below the horizontal diameter, and uniformly distributed loads on the top quarter and bottom quarter of the circumference of the pipe.

The bending moment at the top and bottom of a pipe of diameter d is $0.159Qd$ under a concentrated load, Q; $0.0625Wd$ under a total uniformly distributed load, W; $0.0845Wd$ under a total load, W, distributed over the top fourth of the circumference and with the pipe supported on its bottom quarter circumference.

The bending moments at the ends of the horizontal diameters under these conditions of loading are $0.091Qd$, $0.0625Wd$ and $0.077Wd$, respectively.

Pipe have been tested under all three loadings. Some testing by concentrated loading is carried on annually by the Bureau of Sewers of Brooklyn, N. Y. The tests at the Iowa Engineering Experiment Station are made by loading a fourth of the pipe's circumference, as this loading seemed to Marston and Anderson to reproduce better than any other the conditions in a backfilled trench. The leading published collections of American test results are those of F. A. Barbour in *Jour. Assoc. Eng. Socs.*, Dec. 1897, Marston and Anderson in Bulletin 31, Iowa Engineering Experiment Station, and M. A. Howe, in *Jour. Assoc. Eng. Socs.*, June, 1891, the last being summarized in Table 112.

In 1890 tests were made at the Rose Polytechnic Institute, Terre Haute, Ind., by Prof. M. A. Howe, on pipe from 15 manufacturers.

TABLE 112.—HYDROSTATIC TESTS OF VITRIFIED CLAY PIPE MADE AT ROSE POLYTECHNIC INSTITUTE BY PROF. M. A. HOWE

Designation of manufacturer		A	B	C	D	E	F	G	H	I	J	K	L	M	N	P
Average tensile strength by method	1	......	245.8	476.0	376.0	......	......	......	......	......	......	......	......	......	......	......
	2	582.8	359.5	424.0	455.7	470.8	727.3	271.0	......	......	......	......	......	......	......	......
	3	814.0	616.3	435.8	744.7	......	......	252.0	665.3	427.4	547.7	647.7	939.0	943.5	1081.8	618.6
No. of tests........		8	12	8	13	1	3	7	12	7	6	13	6	4	5	11
Average tensile strength (lb. per sq. in.)...........		669.5	407.2	442.9	582.9	470.8	727.3	265.6	665.3	427.4	547.7	647.7	939.0	943.5	1081.8	618.6
Minimum gage pressure (lb. per sq. in.).............		50	28	47	25	62	125	12	76	42	63	59	144	139	85	37
Minimum tensile strength (lb. per sq. in.).........		265	142	261	81	241	714	68	253	231	329	288	800	836	530	192
Maximum tensile strength (lb. per sq. in.)..........		1054	775	580	921	988	745	507	1023	587	751	1099	1093	1095	1825	1086

The results were published in the *Journal of the Association of Engineering Societies*, June, 1891, and are summarized in Table 112. Three different methods were used in making the hydrostatic tests. The first and second caused a pressure to be exerted upon the ends of the pipe, while in the third no pressure was brought to bear on the ends, which were closed with leather cups. The tests showed consistently that the third method gave higher results, and it was considered more reliable by Prof. Howe. The other methods gave lower results because of stresses due to end pressure. The averages of tensile strengths for the different brands of pipe varied from 265.6 to 1081.8 lb. per square inch, while the general average of all results was 600.4 lb. The minimum recorded gage pressure was 12 lb. per square inch; the minimum tensile strength 68 lb. per square inch; and the maximum tensile strength 1825 lb., each of these being for a single test.

"In the hydrostatic tests the color of the fracture, with hardly an exception, was the criterion of strength, each class having its particular color corresponding to the greatest strength."

TABLE 113.—MEAN MINIMUM AND MAXIMUM AND AVERAGE RESISTANCE TO INTERNAL PRESSURE OF GERMAN VITRIFIED CLAY PIPE

(Burchartz and Stock in *Eng. Rec.*, Aug. 18, 1906)

Average internal diameter (inches)	Average thickness of shell (inches)	Number of tests	Internal water pressure in atmospheres			Ultimate average tangential stress (pounds per square inch)
			Average minimum	Average maximum	Total average	
2	0.72	3	24.0	28.0	26.1	490
3	0.80	3	15.0	18.4	16.3	490
4	0.68	11	9.2	25.2	20.1	830
6[1]	0.80	23	8.3	24.5	17.2	850
8	0.92	13	14.6	24.2	17.9	1,120
10	0.96	2	6.0	8 0	7.0	600
12	1.04	19	6.7	16.9	11.9	920
16	1.12	2	9.4	9.7	9.6	910
18	1.28	8	7.1	12.0	9.3	920
20	1.44	4	9.0	12.2	10.6	1,040
24	1.72	7	5.3	8.6	7.2	720
28	1.88	4	7.1	10.2	8.7	910
32	1.96	14	5.4	9.0	7.7	1,120
Average......	....	....	....	....	13.9	850

[1] Includes pipes of 0.62 and 0.64 in. diameter. One atmosphere = 14.697 lb. per square inch = 33.9 ft. of water. Minimum pressure, 5.3 atm. = 180 ft. water = 78 lb. per square inch.

Tests were made at the Royal Testing Laboratory in Berlin, by Messrs. Burchartz and Stock, and their results for 1896–1904 inclusive were given in *Engineering Record*, August 18, 1906. The mean minimum and maximum and average values for resistance against internal

pressure are given in Table 113. The average water pressure was 13.9 atmospheres or 204 lb. per square inch, and the average ultimate tensile strength 650 lb. per square inch. The minimum pressure was 5.3 atmospheres, equivalent to 78 lb. per square inch, or 183 ft. head of water. These results from German pipe are higher than those obtained by Prof. Howe from American pipe.

Although vitrified pipe may be manufactured of such strength as to stand hydrostatic pressure of 100 lb. per square inch, or even more, it is a question whether the joints now in use will be equally strong. There are few published data on this subject. The tests made by Prof. Howe at the Rose Polytechnic Institute on cement joints are summarized in Table 114. These tests are rather unsatisfactory, because of the wide range of the results. It would appear, however, that the pressure which the joints withstood is much less than the pressure which the pipe was capable of holding. It is rather hard to understand why the pipe broke in several cases instead of the joints failing, but this may be partly due to the methods of testing.

TABLE 114.—HYDROSTATIC TESTS OF WELL-MADE NATURAL CEMENT JOINTS FOR VITRIFIED CLAY PIPE—(HOWE)

Nominal internal diameter of pipe (inches)	Average thickness of cement joint (inches)	Average depth of cement joint (inches)	Proportions used for cement joint (inches)	Age of joint (days)	Test method employed	Pressure sure, (pounds per square inch)	Remarks
6	0.26	1.40	neat	33	2	148.0	Pipe broke.
6	0.44	1.65	1 : 2	14	2	none	Joint failed.
6	0.34	1.63	neat	21	3	25.0	Joint failed.
6	0.34	1.75	neat	21	3	17.5	Pipe broke.
6	0.30	1.80	1 : 1	6	3	25.3	Joint failed.
8	0.35	1.50	neat	41	2	50.0	Pipes broke.
8	0.28	1.70	neat	16	1	50.0	Pipes broke.
8	0.50	1.80	neat	21	3	none	Joint failed.
8	0.27	1.78	neat	21	3	17.5	Joint failed.
8	0.56	1 90	1 : 1	6	3	12.0	Joint failed.
10	0.35	1.40	neat	21	1	none	Pipe broke.
12	1.50	4.50	neat	33	3	115.0	Joint leaked.
12	0.25	1.75	neat	21	3	6.0	Joint failed.
12	0.44	1.85	neat	28	3	17.5	Joint failed.
12	0.50	1.83	neat	21	3	12.0	Pipe broke.
12	0.24	3 05	neat	6	3	37.5	Pipe broke.

Mr. Barbour stated, as a result of his tests, that the thickness of pipe when stressed to its ultimate strength was

$$t = \sqrt[1.65]{\frac{pd}{c}}$$

where t was the thickness in inches, p the pressure in pounds per linear inch, d the internal diameter in inches, and c a constant taken empirically at 33,000.

Professor Talbot deduced his formulas for thickness from the expressions for the maximum bending moments. These are $t = 0.976\sqrt{(Qd/f)}$ for concentrated vertical loading, and $t = 0.25\sqrt{(6Wd/f)}$ for a uniformly distributed vertical load, where f is the unit stress in the remotest fiber.

Marston and Anderson give for the 90 deg. top loading and 90 deg. bottom support $t = \sqrt{(0.5Wd/f)}$.

The modulus of rupture in many lots of sewer pipe tested by Marston at Ames ranged from 910 to 1940 lb. per square inch for single-strength and 790 to 1720 lb. for double-strength pipe. In these tests, as in those of F. P. Johnson (*Eng. News*, March 19, 1886), it is apparent that the modulus of rupture so obtained is two or three times the tensile strength of the material. The same is true of the results of tests of small cement pipe, in which the modulus of rupture was as high as 1000 lb. per square inch in many cases.

Resistance of Cast-iron Pipe to Internal Pressure.—The standard specifications of the New England Water Works Association, adopted in 1902, were based primarily upon the practice of the Metropolitan Water Works of Boston. The thickness of the pipe was determined by the formula

$$t = \frac{(p+p')\,r}{3300} + 0.25, \text{ in which}$$

t = thickness in inches,
p = static pressure in pounds per square inch,
p' = pressure due to water hammer in pounds per square inch,
r = internal radius of pipe, in inches.
3300 = $\frac{1}{5}$ tensile strength of cast iron, taken to be 16,500 lb. per square inch,
0.25 = allowance for deterioration by corrosion and other causes.

Values given to p' as follows:

Diameter of pipe	p' in pounds per sq. in.
4, 6, 8 and 10 in.	120
12 and 14 in.	110
16 and 18 in.	100
20 in.	90
24 in.	85
30 in.	80
36 in.	75
42 to 60 in.	70

It will be noted that the allowance for water hammer is a very substantial one.

TABLE 115.—STANDARD THICKNESSES AND WEIGHTS OF CAST-IRON PIPES
(New England Water-works Association)
(Laying Length 12 ft.)

Nominal diam. of pipe	Class A		Class B		Class C		Class D		Class E		Class F		Class G		Class H		Class I		Class K	
	Thickness of shell, inches	Weight per length, pounds	Thickness of shell, inches	Weight per length, pounds	Thickness of shell, inches	Weight per length, pounds	Thickness of shell, inches	Weight per length, pounds	Thickness of shell, inches	Weight per length, pounds	Thickness of shell, inches	Weight per length, pounds	Thickness of shell, inches	Weight per length, pounds	Thickness of shell, inches	Weight per length, pounds	Thickness of shell, inches	Weight per length, pounds	Thickness of shell, inches	Weight per length, pounds
4	0.34	200	....	....	0.36	215	....	....	0.39	230	....	....	0.42	250	....	....	0.45	265	0.48	280
6	0.38	330	....	....	0.42	350	....	....	0.46	380	....	....	0.50	420	....	....	0.54	445	....	....
8	0.42	475	....	....	0.48	530	....	....	0.53	575	....	....	0.58	640	....	....	0.63	690	....	....
10	0.47	650	0.50	680	0.53	720	0.56	760	0.60	810	0.63	850	0.67	890	0.70	935	....	....	....	....
12	0.49	810	0.53	855	0.57	910	0.61	970	0.65	1,040	0.69	1,100	0.73	1,160	0.77	1,220	....	....	....	....
14	0.53	1,010	0.57	1,080	0.61	1,150	0.66	1,220	0.70	1,310	0.75	1,390	0.79	1,460	0.83	1,530	....	....	....	....
16	0.55	1,215	0.60	1,300	0.65	1,390	0.70	1,490	0.75	1,610	0.80	1,710	0.85	1,810	0.90	1,900	....	....	....	....
18	0.57	1,400	0.63	1,520	0.69	1,660	0.75	1,780	0.80	1,910	0.86	2,040	....	....	....	....	....	....	....	....
20	0.60	1,610	0.66	1,760	0.72	1,920	0.79	2,090	0.85	2,260	0.92	2,420	....	....	....	....	....	....	....	....
24	0.64	2,050	0.72	2,290	0.80	2,550	0.88	2,780	0.95	3,000	1.03	3,240	....	....	....	....	....	....	....	....
30	0.71	2,860	0.81	3,230	0.91	3,600	1.01	3,950	1.10	4,340	1.20	4,700	....	....	....	....	....	....	....	....
36	0.79	3,800	0.90	4,270	1.02	4,840	1.13	5,310	1.25	5,900	1.37	6,400	....	....	....	....	....	....	....	....
42	0.87	4,920	1.00	5,560	1.13	6,270	1.27	6,970	1.40	7,720	1.53	8,360	....	....	....	....	....	....	....	....
48	0.95	6,130	1.10	6,970	1.25	7,920	1.40	8,780	1.55	9,740	1.70	10,600	....	....	....	....	....	....	....	....
54	1.03	7,510	1.20	8,600	1.37	9,800	1.54	10,900	1.72	12,400	1.90	13,500	....	....	....	....	....	....	....	....
60	1.10	8,900	1.30	10,300	1.50	11,900	1.70	13,300	1.90	15,100	2.10	16,500	....	....	....	....	....	....	....	....

These specifications also require a hydrostatic test, under which "the pressures to which the different sizes and classes of pipe shall be subjected" are as follows:

(Page 95, *Journ. N. E. W. W. Assn.*, 1902)
Class A pipe, 150 lb. per square inch.
Class B pipe, 200 lb. per square inch.
Class C pipe, 250 lb. per square inch.
Class D pipe, 300 lb. per square inch.
Class E pipe, 350 lb. per square inch.
Class F pipe, 400 lb. per square inch.

The specifications do not indicate the conditions under which the different classes of pipe can be used, this being left to the judgment of the engineer, as being dependent not only upon the pipe pressures to be met, but also the environment of the pipe. It may be added, however, that Dexter Brackett, Chief Engineer of the Metropolitan Water Works, who was largely responsible for the development of this standard, states that in the practice of the Metropolitan Water Board, which

TABLE 116.—STANDARD WEIGHTS PER FOOT OF STRAIGHT PIPE, EXCLUSIVE OF SOCKETS
(New England Water-works Association)

Nominal diam.	Class	Weight per foot in lb.	Nominal diam.	Class	Weight per foot in lb.	Nominal diam.	Class	Weight per foot in lb.	Nominal diam.	Class	Weight per foot in lb.
4	A	14.89	12	C	70.67	18	E	148.4	36	F	502 0
4	C	15.70	12	D	75.39	18	F	159.0	42	A	368.4
4	E	16.92	12	E	81.99	20	A	121.9	42	B	422.1
4	G	18.89	12	F	86.77	20	B	133.7	42	C	481.1
4	I	20.10	12	G	91.51	20	C	147.6	42	D	538 9
4	K	21.30	12	H	96.22	20	D	161.4	42	E	600.6
6	A	24.32	14	A	76.85	20	E	175.6	42	F	651.4
6	C	26.72	14	B	82.41	20	F	189.5	48	A	450.3
6	E	29.08	14	C	87.97	24	A	155 6	48	B	530.2
6	G	32.40	14	D	94.85	24	B	174.4	48	C	608.0
6	I	34 79	14	E	102.73	24	C	196.3	48	D	678 9
8	A	35.58	14	F	109 70	24	D	215.3	48	E	758 5
8	C	40.38	14	G	115.24	24	E	234.5	48	F	829 4
8	E	44.33	14	H	120.74	24	F	253.5	54	A	559.8
8	G	49.65	16	A	90.98	30	A	215.3	54	B	650.3
8	I	53 62	16	B	98.95	30	B	244.8	54	C	749.5
10	A	49 04	16	C	106.9	30	C	277.7	54	D	839.9
10	B	52.03	16	D	114.8	30	D	307.3	54	E	946 9
10	C	54.99	16	E	125.5	30	E	338.0	54	F	1042.7
10	D	57.94	16	F	133.5	30	F	367.5	60	A	664.0
10	E	63.61	16	G	141.4	36	A	287.0	60	B	782.3
10	F	66.61	16	H	149.3	36	B	326.0	60	C	911.5
10	G	70.57	18	A	104.5	36	C	373.3	60	D	1029.7
10	H	73.53	18	B	115.2	36	D	412.3	60	E	1162.0
12	A	61.14	18	C	127.4	36	E	459.6	60	F	1280.0
12	B	65.92	18	D	138 0						

TABLE 117.—WEIGHTS OF CAST-IRON PIPE ACCORDING TO DIFFERENT AUTHORITIES

Diam. of pipe, inches	William Wheeler				Freeman C. Coffin[1]								N. Henry Crafts				N E. W. W. A. Com., 1902					
	General		Heavy		Light		Standard		Heavy		Lightest		Light, 65 lb. pressure		Heavy, 150 lb. pressure		Class A		Class C		Class F	
	Per foot	Per 12 ft. length	Per foot	Per 12 ft. length	Per foot	Per 12 ft. length	Per foot	Per 12 ft. length	Per foot	Per 12 ft. length	Per foot	Per 12 ft. length	Per foot	Per 12 ft. length	Per foot	Per 12 ft. length	Per foot	Per 12 ft. length	Per foot	Per 12 ft. length	Per foot	Per 12 ft. length
3	14	168	14	168	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
4	17	204	18	216	17	204	19	228	22	264	16.4	197	18	216	18	216	16.7	200	17.7	212	19.2	230
6	28	336	29	348	27	324	30	360	35	420	26.3	315	28	336	28	336	27.6	330	29.6	355	33.3	400
8	40	480	42	504	40	480	45	540	50	600	36.7	440	39	468	44	528	39.6	475	43.8	525	50.0	600
10	53	636	57	684	54	648	60	720	70	840	50.0	600	52	624	65	780	54.3	650	60.4	725	70.4	845
12	67	804	74	888	68	816	80	960	90	1,080	64.1	768	65	780	87	1,044	67.5	810	76.7	920	90.0	1,090
14	.....	1,116	93	1,116	85	1,020	100	1,200	115	1,380	79.4	952	80	960	113	1,356	83.4	1,010	96.5	1,155	115.0	1,380
16	.....	1,368	114	1,368	105	1,260	125	1,500	145	1,740	101.2	1,215	96	1,152	143	1,716	101.2	1,215	117.5	1,410	141.6	1,700
18	.....	....	137	1,644	120	1,440	150	1,800	175	2,100	114.2	1,370	.....	.....	.....	.....	117.5	1,410	138.3	1,660	160.0	2,040
20	.....	....	162	1,944	140	1,680	175	2,100	205	2,460	133.6	1,603	134	1,608	213	2,556	134.2	1,610	160.0	1,920	200.8	2,410
24	.....	..	.....	.....	185	2,220	225	2,700	270	3,240	...	....	175	2,100	298	3,576	170.8	2,050	210.8	1,530	270.0	3,240

[1] See *Journal New England Water-works Association*, September, 1900. "Light," safe for 100-ft. head; "Standard," safe for 350-ft. head; "Heavy," for city use and higher heads than 350 ft.

TABLE 118.—STANDARD THICKNESS AND WEIGHTS OF CAST-IRON PIPE
(American Water-works Association)

Nominal inside diameter, inches	Class A, 100-ft. head, 43 lb. pressure			Class B, 200-ft. head, 86 lb. pressure			Class C 300-ft. head, 130 lb. pressure			Class D, 400-ft. head, 173 lb. pressure			Nominal inside diameter, inches
	Thickness, inches	Weight per Foot	Weight per Length	Thickness, inches	Weight per Foot	Weight per Length	Thickness, inches	Weight per Foot	Weight per Length	Thickness, inches	Weight per Foot	Weight per Length	
4	0.42	20.0	240	0.45	21.7	260	0.48	23.3	280	0.52	25.0	300	4
6	0.44	30.8	370	0.48	33.3	400	0.51	35.8	430	0.55	38.3	460	6
8	0.46	42.9	515	0.51	47.5	570	0.56	52.1	625	0.60	55.8	670	8
10	0.50	57.1	685	0.57	63.8	765	0.62	70.8	850	0.68	76.7	920	10
12	0.54	72.5	870	0.62	82.1	985	0.68	91.7	1,100	0.75	100.0	1,200	12
14	0.57	89.6	1,075	0.66	102.5	1,230	0.74	116.7	1,400	0.82	129.2	1,550	14
16	0.60	108.3	1,300	0.70	125.0	1,500	0.80	143.8	1,725	0.89	158.3	1,900	16
18	0.64	129.2	1,550	0.75	150.0	1,800	0.87	175.0	2,100	0.96	191.7	2,300	18
20	0.67	150.0	1,800	0.80	175.0	2,100	0.92	208.3	2,500	1.03	229.2	2,750	20
24	0.76	204.2	2,450	0.89	233.3	2,800	1.04	279.2	3,350	1.16	306.7	3,680	24
30	0.88	291.7	3,500	1.03	333.3	4,000	1.20	400.0	4,800	1.37	450.0	5,400	30
36	0.99	391.7	4,700	1.15	454.2	5,450	1.36	545.8	6,550	1.58	625.0	7,500	36
42	1.10	512.5	6,150	1.28	591.7	7,100	1.54	716.7	8,600	1.78	825.0	9,900	42
48	1.26	666.7	8,000	1.42	750.0	9,000	1.71	908.3	10,900	1.96	1,050.0	12,600	48
54	1.35	800.0	9,600	1.55	933.3	11,200	1.90	1,141.7	13,700	2.23	1,341.7	16,100	54
60	1.39	916.7	11,000	1.67	1,104.2	13,250	2.00	1,341.7	16,100	2.38	1,583.3	19,000	60
72	1.62	1,283.4	15,400	1.95	1,545.8	18,550	2.39	1,904.2	22,850	......	......	......	72
84	1.72	1,633.4	19,600	2.22	2,104.2	25,250	......	......	......	......	......	......	84

The above weights are per length to lay 12 ft., including standard sockets; proportionate allowance to be made for any variation.

supplies water to Boston and other cities and towns within a radius of ten miles, Class A pipe of the N. E. Water Works Assn. standard specifications (Tables 115 and 116) has been used for static heads of 50 ft., Class B for 100 ft., etc., each class advancing about 50 ft.

In the case of works supplying smaller cities or towns, where excavations in the streets are of less frequent occurrence, these classes of pipe have been used under much heavier pressures than those in accord with the Boston practice, 10-in. Class A pipe having been used successfully under static pressures as high as 125 lb. The past practice of certain engineers in this respect, covering a period of from 20 to 30 years of successful experience with the pipes of the stated weights, is shown in Table 117.

In sewerage work the lightest weight classes of the New England Water Works Association standard specification pipe are generally used, except in such cases as may involve very heavy traffic loads and pressures, and probability of future displacement from one cause or another.

In 1908, the American Water Works Association adopted somewhat similar specifications for cast-iron pipe, but with radically different weights for the different classes, as shown in Table 118.

MANUFACTURE OF CLAY AND CEMENT PIPE

The methods of manufacturing cement and vitrified pipe differ as radically as do the properties of these two classes of sewer materials. The distinction between the two is so complete, in fact, that the attempt to lay down standard requirements applicable to both alike has not yet been successful, and the most satisfactory results have been obtained when specifications are prepared for each class of pipe, independently of the other, in such a way as to utilize to the full the useful properties of the materials and technical processes employed in the industry.

Vitrified Clay Pipe.—Vitrified salt-glazed pipe are made from clays and shales, only a small part of these raw materials found in nature being fit for the purpose. They are prepared in various ways, according to their source, for the press which forms them, the methods required for shale obviously being different from those for clay. This stage of the manufacturing process is somewhat important because some of the surface pimpling on salt-glazed pipe is apparently due to the imperfect preparation of the shale or clay as well as to the heat treatment. A. J. Aubrey reported in *Trans. Am. Ceramic Soc.*, 1907, that by passing all clay through a 16-mesh screen he was able to effect a marked reduction in the pimpling as compared with the results when an 8-mesh screen was used. A 29-mesh screen was little more effective than a 16-mesh. He also found that the pimples were apparently caused by the incipient

fusing, bubbling and swelling of small particles of shale, lying close to the surface of the pipe, although other makers reported that the pimples on their product seemed to be due to the oxidation of the iron in the clay during the burning. The general opinion of those discussing the subject at that time was that pimples could be avoided to a large extent by attention to temperature regulation during the burning and by glazing only when the flame had become perfectly clear..

The prepared clay is placed in the hopper of a press, this hopper being a cylinder 24 in. or so in diameter, with the wall drawn in at the bottom to the shape of the outside of the bell of a pipe. A rod is held in

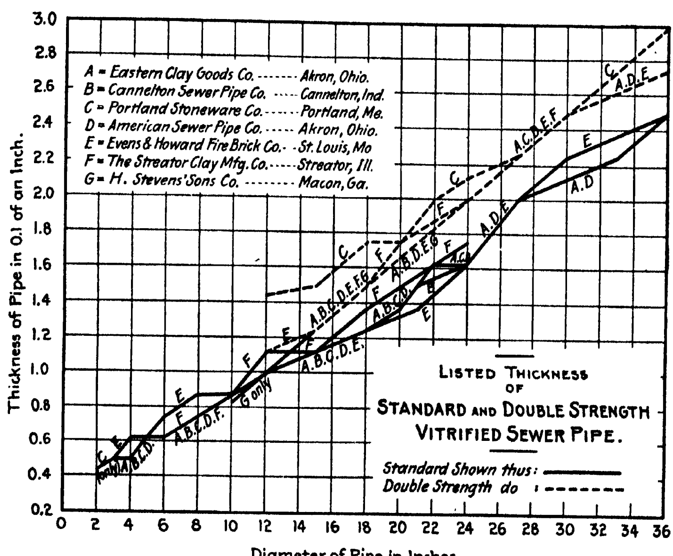

FIG. 122.—Listed thickness of standard and double-strength vitrified sewer pipe.

in the axis of the cylinder by a spider and to its bottom is attached a core die or bell, at the elevation where the wall of the cylinder is drawn in. An annular space as thick as the green shell of the pipe is left in this way in the bottom of the hopper, and the pipe is formed by pressing the clay through this space, the inside of the bell being formed by a mold on the top of a moving platform on which the green pipe is lowered. The thickness of straight and curved pipe is given in Fig. 122 and Table 119.

The pipe are generally seasoned under cover for some time, to allow as much water to evaporate as will naturally pass off in this way. The

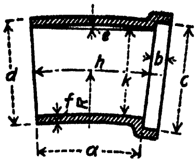

TABLE 119.
DIMENSIONS OF CURVED VITRIFIED CLAY PIPE

k in.	a deg.	a min.	b in.	c in.	c in.	d in.	e in.	f in.	h in.	R-radius ft.
6	11	28	$2\frac{1}{2}$	$8\frac{1}{2}$	$9\frac{1}{2}$	$7\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	24	10
8	11	28	$2\frac{3}{4}$	$10\frac{1}{2}$	$11\frac{1}{2}$	$9\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	24	10
9	11	28	$2\frac{3}{4}$	$11\frac{5}{8}$	$12\frac{5}{8}$	$10\frac{5}{8}$	$\frac{5}{8}$	$\frac{13}{16}$	24	10
10	11	28	$2\frac{3}{4}$	$12\frac{3}{4}$	$13\frac{3}{4}$	$11\frac{3}{4}$	$\frac{5}{8}$	$\frac{7}{8}$	24	10
12	11	28	3	15	16	14	$\frac{5}{8}$	1	24	10
15	11	28	3	$18\frac{1}{2}$	$19\frac{1}{2}$	$17\frac{1}{2}$	$\frac{5}{8}$	$1\frac{1}{4}$	24	10
18	11	28	$3\frac{1}{4}$	22	23	21	$\frac{5}{8}$	$1\frac{1}{2}$	24	10
20	11	28	$3\frac{1}{2}$	$24\frac{1}{2}$	$25\frac{1}{2}$	$23\frac{1}{2}$	$\frac{5}{8}$	$1\frac{3}{4}$	24	10
24	11	28	4	29	30	28	$\frac{11}{16}$	2	24	10
30	11	28	$4\frac{1}{2}$	36	37	35	$\frac{11}{16}$	$2\frac{1}{2}$	24	10
36	11	28	5	$42\frac{1}{2}$	$43\frac{1}{2}$	$41\frac{1}{2}$	$\frac{11}{16}$	$2\frac{3}{4}$	24	10
6	5	44	$2\frac{1}{2}$	$8\frac{1}{2}$	$9\frac{1}{2}$	$7\frac{1}{2}$	$\frac{5}{16}$	$\frac{3}{4}$	24	20
8	5	44	$2\frac{3}{4}$	$10\frac{1}{2}$	$11\frac{1}{2}$	$9\frac{1}{2}$	$\frac{5}{16}$	$\frac{3}{4}$	24	20
9	5	44	$2\frac{3}{4}$	$11\frac{5}{8}$	$12\frac{5}{8}$	$10\frac{5}{8}$	$\frac{5}{16}$	$\frac{13}{16}$	24	20
10	5	44	$2\frac{3}{4}$	$12\frac{3}{4}$	$13\frac{3}{4}$	$11\frac{3}{4}$	$\frac{5}{16}$	$\frac{7}{8}$	24	20
12	5	44	3	15	16	14	$\frac{5}{16}$	1	24	20
15	5	44	3	$18\frac{1}{2}$	$19\frac{1}{2}$	$17\frac{1}{2}$	$\frac{5}{16}$	$1\frac{1}{4}$	24	20
18	5	44	$3\frac{1}{4}$	22	23	21	$\frac{5}{16}$	$1\frac{1}{2}$	24	20
20	5	44	$3\frac{1}{2}$	$24\frac{1}{2}$	$25\frac{1}{2}$	$23\frac{1}{2}$	$\frac{5}{16}$	$1\frac{3}{4}$	24	20
24	5	44	4	29	30	28	$\frac{5}{16}$	2	24	20
30	5	44	$4\frac{1}{2}$	36	37	35	$\frac{5}{16}$	$2\frac{1}{2}$	24	20
36	5	44	5	$42\frac{1}{2}$	$43\frac{1}{2}$	$41\frac{1}{2}$	$\frac{5}{16}$	$2\frac{3}{4}$	24	20

pipe are then placed in the kiln, work requiring considerable skill based on long experience in order to reduce the number of inferior pipe to a minimum. The burning of a kiln is a slow process and it is during this that most of the defects of sewer pipe appear. The irregular zigzag "fire cracks," usually circumferential, are apparently caused by the direct play of either hot gases or cool air on the surface of the pipe where they are found. Other cracks appear as a network over a large part of the surface of a pipe and are usually attributed to heating the pipe too rapidy in the early stages of the burning. In passing through the die, there seems to be a tendency for the clay to become laminated, and when a network of these so-called "water cracks" is formed, they are not

likely to extend deeply into the shell, on account of this lamination, unless the pipe are large and the heating is conducted vigorously from the beginning of the burning, in which case the gases in the clay may be generated so rapidly that large blisters form on the surface of the pipe and a few lengths may have flakes blown from their surface.

Vitrification brings these defects out more prominently than they appear before that stage in the process is reached, because with vitrification there comes a tendency toward shrinkage.

These cracks are the main defects in vitrified sewer pipe. When shallow they have little appreciable effect on their strength, and for this reason, Emil Kuichling, after a careful study of pipe making and the requirements for satisfactory service in sewers, recommended in widely quoted specifications for use at Rochester, that one fire crack not more than 1/8 in. wide should not cause the rejection of a length of pipe, provided; first, if it went through the shell, that it was not over 2 in. long when at the spigot or 1 in. long at the bell; second, if it went through only two-thirds of the thickness of the pipe, that it was not over 4 in. long; third, if it went through only one-half of the thickness of the shell, that it was not over 6 in. long; fourth, if it went through less than one-half of the thickness of the shell, that it was not over 8 in. long; fifth, if it was a transverse crack, that it was not longer than one-sixth of the circumference of the pipe. Two or more fire cracks of any of these five classes in one length of pipe was cause for rejection. Irregular lumps and unbroken blisters on the inside of a pipe, when not more than 1/4 in. high and 1 or 2 in. in diameter, were not considered a sufficient obstacle to the flow of the sewage to justify rejecting a pipe on which they appeared, the rule being to reject a pipe or special having on its inner surface a broken blister or flake thicker than one-sixth the standard thickness of the shell and longer than one-twelfth of the inner circumference of the pipe, and to reject it anyway if the pipe could not be laid so as to have the blister on top. So far as warping during burning was concerned, Kuichling's specifications required the bells to leave without chipping a space of at least 1/8 in. around a spigot inserted in it. At least 60 per cent. of all pipe less than 12 in. in diameter had to be substantially circular and at least 40 per cent. of those 12 in. or more in diameter. In no case must the long diameter of an accepted pipe be more than 6 or 7 per cent. longer than the short diameter.

The shape of the bell of sewer pipe was investigated in the winter of 1911-1912, by the Institution of Municipal and County Engineers of Great Britain, which appointed a Standardization Committee to determine the best type of socket. The great objection to the old type, with the interior of the bell parallel to the shell of the pipe, is that the workmen did not always center the spigot of the pipe they were laying accurately in the bell of the pipe just laid, therefore there was a tendency

of the spigot to be low in the bell thus producing a roughness at each joint. To overcome this objection, a type of socket was recommended by the committee which has walls flaring out like the sides of a funnel, so that when the spigot of a pipe is introduced in the bell of the preceding pipe laid, the two sections are lined up without any attention from the workmen and a satisfactory surface is obtained along the invert. The committee was of the opinion that this form of joint would also prove desirable on account of the smaller amount of material which was necessary to fill the joint space, and the reduction of the chance that something might enter the pipe during the making of the joint.

Cement Pipe.—The amount of capital required to put up a small plant for making cement tile and pipe is so moderate that a large number of these little works have been built. Owing mainly to lack of skill, working capital, or both, much inferior pipe has been produced in these small plants, and this poor product has prejudiced many engineers against all cement pipe. There are a number of large cement pipe plants, however, each representing a considerable investment and some managed by technically educated engineers, where pipe of fairly uniform grade are produced. These works are ready at all times to submit their product to comprehensive tests like those adopted about 1911 by the Iowa Society of Engineers and other technical and drainage organizations in that state. The existence of widely recognized standard requirements of a fair yet rigid character and frequent tests to insure the rejection of products not meeting these specifications are necessary factors in any satisfactory condition of the cement pipe industry; without them a product having so little uniformity as to be positively unreliable is likely to flood the market.

The materials for cement pipe must be of high grade, particularly when the pipe are made from a dry mix. Cement which will pass the specifications of the American Society for Testing Materials will prove satisfactory for pipe manufacture in most cases, although there is a very slight chance that a brand may be found occasionally which will not work as well with local sand as other brands which test no better. The sand and gravel or broken stone, which must be clean, are generally made up, in the best plants, in proportions which give approximately a 1 to 4 mix for sizes up to 20 in. to be used under heads not exceeding 15 ft. There is a great variation, however, in this practice, and a 1 to 3 mix is, perhaps, more customary in pipe above 20 in. The individual preferences of purchasers, as well as manufacturers, have prevented anything like uniformity being reached, and this is also true of the use of a small amount of hydrated lime in the mix, which was widely practised in Southern California about 1900, for instance. An extensive series of tests was made by George P. Dieckmann, chief chemist of the Northwestern States Portland Cement Co., to determine the best mix

for pipe, and the results led him to recommend for the fine aggregate a material so graded that not more than 10 per cent. would remain on a 10-mesh sieve, nor more than 30 per cent. pass a 50-mesh sieve. The practice regarding the gravel or broken stone forming the coarse aggregate seems to be, in the case of tile, to require all this material which is used to be retained on a screen with 1/4-in. holes and to be not more than half the thickness of the wall of the pipe. Small pipe up to about 10 in. in diameter are preferably made without any coarse aggregate, for experience shows that it is extremely difficult to produce a dense uniform material from a mix containing coarse aggregate when used in walls as thin as those of small pipe.

Two methods of manufacturing cement pipe are employed, known as the "dry" and the "slush" methods, respectively. The dry method is essentially the same as that of the concrete block industry. A mixture is employed which contains only enough water to leave web-like markings on the surface of the concrete when the forms are removed, and to ball up when pressed in the hand. The density of the pipe depends on the thoroughness of the ramming of the materials into the molds as well as on the character of the mixing. The dry-mixed pipe also require careful curing. As a result of these requirements for satisfactory pipe, the dry method of manufacture gives most satisfaction when conducted in a plant with adequate mechanical facilities and storage room. The slush method of manufacture is usually followed with the large sizes of pipe. The material is made up into a mixture so wet that with a small amount of ramming it will flow into every part of the mold. The pipe must remain in the forms longer than in the case of dry-mixed pipe, but when removed these pipe require less curing than the dry-mixed product. The slush process is advantageously coupled with steam curing by the Lock Joint Pipe Co., which has found that the quality of the product, particularly the density of its surface, is much better if the pipe is steamed both before and after it is taken from the molds.

The smaller sizes of pipe, such as are carried in stock for general sale, are made on machines of two general types. In the first, the space between the two shells forming the pipe mold is gradually filled with the mix, and tamped by an apparatus which produces approximately the same effect as hand tamping. The second type of machine, used mainly for small sizes, has a revolving head or packer which moves up and down inside the shell or form for the outer surface of the pipe, no inner shell being required in this class of machine.

The pipe produced on these machines, which are all used with a dry mix so as to permit the removal of the mold as soon as the machine work is finished, are generally placed on narrow-gage cars having three or four decks. These cars are run by hand into curing rooms where the pipe are subjected to one of treatments. In a few plants the pipe are

removed to these rooms on belt conveyors and in the small works of low first cost this part of the manufacturing process is carried on by hand.

The curing is done either with low-pressure steam or by spraying water over the pipe from a hose. The steam is usually employed at a pressure of about 5 lb. and so regulated that the steam chamber will be kept at a temperature of 70° to 120° F. With the average dry mix, the pipes are left in this damp atmosphere for 48 hours, although in very cold weather some makers leave the pipe a day longer, before sending it out to the yard. It is generally considered desirable for the pipe to stay in the yards at least two weeks before being shipped from the works.

The second, or natural, method of curing is generally carried out by sprinkling the pipe as it rests on the shelves in the curing rooms with water from a hose. This has to be done rather carefully while the pipe are green, for it is possible to injure them by allowing a stream of some size to strike them directly. The sprinkling is carried on three or four times a day from the time the pipe are hard enough to stand the treatment until they become well hardened, which may be anywhere from 6 to 8 days, depending on atmospheric conditions. The pipe hardened in this way are usually kept in the storage yards at least 24 days before being shipped. The general opinion seems to be that steam curing, when well conducted, not only gives a more uniform product but is also more economical. A few manufacturers who practise it sprinkle the pipe sent from the steaming rooms to the yard, for two or three days after they are in storage.

A large amount of cement pipe, about 400 miles in 1912, has been laid in Brooklyn, N. Y., and most of this was furnished by the Wilson & Baillie Manufacturing Co., of that borough. The methods of manufacture were described substantially as follows, by Gustave Kaufman, the company's engineer, in a paper read before the National Association of Cement Users in 1912.

The pipe are made in 6-, 9-, 12-, 15-, 18- and 24-in. sizes, the 6 and 9 in. being plain round pipe and the others of equivalent capacities to the round pipe of the same diameter. They are 3 ft. in length, with hub joints, with the exception of the 6 in. which is 2 ft. 3 in. The 12-in. pipe is round, with a flat base, and the 15-, 18- and 24-in. pipe are egg-shaped, with flat bases. The thickness of the walls ranges from 3/4 in. for a 6-in. pipe to 2 in. for a 24-in. pipe while the collars have a corresponding variation, ranging from a depth of 1-5/16 in. and a 1/8-in. joint for a 6-in. collar, to a depth of 1-3/4 in., and a 1/4-in. joint for a 24-in. collar.

The cement, sand and trap rock are measured and thoroughly mixed in a machine, evenly fed to the molds, and rammed mechanically with iron rammers regulated to produce continuous and uniform blows of any impact desired.

The machine consists of a mechanical tamper and a revolving table upon which the molds are placed. The tampers have a vertical reciprocating motion and at the same time move outward and inward rapidly so as to conform to the line of the travel of the mold, which, owing to its oval form, presents varying diameters at each revolution to the successive tamping bars. There are eight tool-steel tampers, each making 200 strokes per minute. Only one is down at a time. The head, which consists of the actuating machinery for the tampers, is counter-balanced upward as the mold is being filled with concrete. The head is raised by the density of the concrete and, in this way, an even and regular product is obtained, Mr. Kaufman said. The force of the blow of each rammer is estimated at 800 lb. The area of the arm of the rammer is about 1 sq. in.

The proportions used in recent years are 1-1/2 parts of Portland cement, 1 part of sand and 3 parts of trap rock screenings containing 20 per cent. of stone dust. The quantity of water used to the whole bulk varies from 10 to 15 per cent. according to the condition of the ballast. The mix, when dumped on the floor, is apparently dry, but will ball in the hand under some pressure. A richer mix is used in forming the collar for the reason that as the rammers do not exert a direct blow on the material in the offset, compression of the material cannot be depended upon.

The mixed concrete is delivered to the machines in barrows and is fed into the hoppers by two men, one on either side. As soon as the flask is full and the core automatically lifted clear, the flask is taken up by a pipe truck and wheeled into the stripping rooms where it is allowed to stand usually 30 minutes before it is stripped. After the pipe have set over night a spray of water is turned on and the pipe kept damp for six days, when they are removed from under cover and placed in the yard. The pipe, at the expiration of 30 days, are set sufficiently to be handled in the work.

Spurs for house connections are connected on the pipe. A hole is cut at the proper point on the side of the pipe and a mold is placed in the interior. Cement mortar is then spread over the mold and the connection piece is bedded in place and a heavy band of mortar is wiped around the joint on the outside. After the mortar is removed the inside joint is finished with a trowel.

The method of producing pipe followed by the Colorado Concrete Manufacturing Co. was explained to the authors by Edmond C. van Diest, of Colorado Springs, Colo. as follows:

The mortar is made of Portland cement, sand passing 20-mesh and pit-run sand passing 1/4-in. mesh; for larger pipe, gravel passing 1- to 2-in. mesh is also used. The proportions, by volume, of cement and fine and coarse sand are 4- and 6-in., 1:0.72:1.43 parts; 8-in., 1:0.57

removed to these rooms on belt conveyors and in the small works of low first cost this part of the manufacturing process is carried on by hand.

The curing is done either with low-pressure steam or by spraying water over the pipe from a hose. The steam is usually employed at a pressure of about 5 lb. and so regulated that the steam chamber will be kept at a temperature of 70° to 120° F. With the average dry mix, the pipes are left in this damp atmosphere for 48 hours, although in very cold weather some makers leave the pipe a day longer, before sending it out to the yard. It is generally considered desirable for the pipe to stay in the yards at least two weeks before being shipped from the works.

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The cement, sand and trap rock are measured and thoroughly mixed in a machine, evenly fed to the molds, and rammed mechanically with iron rammers regulated to produce continuous and uniform blows of any impact desired.

pipe from injury while green; the low cost of clay and cement pipe have also tended to discourage this form of construction.

Some work of this class has been done, however, with a traveling mold devised by Ernest L. Ransome and used by him in constructing several sewers and drains. It is shown in Fig. 123. There is an inner core, *A*, of sheet steel about 10 ft. long in the case of an 8-in. pipe. Ahead of this is a shaper, *C*, which trims the bottom of the trench to receive the concrete, and *B* is the mold for the outside surface. The machine is pulled along by a rope, *F*, attached to a deadman set ahead and wound up on the drum, *E*, by the hand lever *D*.

In an account of the use of this machine in Despatch, N. Y., H. P. Gillette stated (*Eng. and Cont.*, March, 1906) that he saw six men and a foreman laying 8-in. pipe at the average rate of 300 ft. in 10 hours. Three men were engaged in mixing and delivering mortar, one in packing mortar into the mold, one in moving it ahead slowly and continuously, and one in placing earth around the green pipe. Where a branch was needed, a hole was cut in the side of the green pipe before the core of the

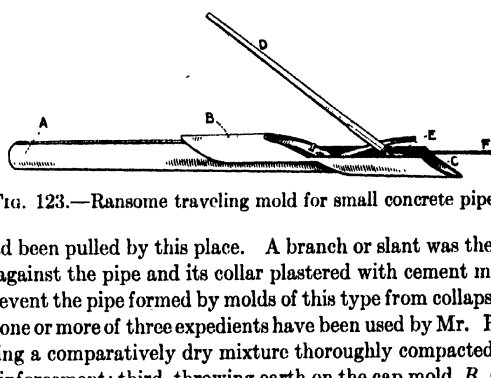

FIG. 123.—Ransome traveling mold for small concrete pipe.

mold had been pulled by this place. A branch or slant was then shoved tightly against the pipe and its collar plastered with cement mortar.

To prevent the pipe formed by molds of this type from collapsing while setting, one or more of three expedients have been used by Mr. Ransome: first, using a comparatively dry mixture thoroughly compacted; second, using reinforcement; third, throwing earth on the cap mold, *B*, and thoroughly compacting it there so as to obtain some arching action in the bottom of the backfill.

Mr. Gillette was of the opinion that the speed attained with the mold would depend very largely on the man who was packing the mortar into the mold, and as this was hard work, it would be advisable to let him change places frequently with the man who worked the lever that pulled the apparatus ahead.

PRACTICAL DEDUCTIONS FROM TESTS AND EXPERIENCE

The investigations described earlier in this chapter show that the loads coming upon pipe are frequently so great that the construction of a tight

sewer under such conditions calls for good, intelligent workmanship. The experiments show that the half elongations of the horizontal diameters of cement and clay pipe do not ordinarily exceed 0.02 in. under breaking loads. It is practically impossible to ram earth around the sides of a pipe so firmly that it will prevent such an insignificant movement, and where the pipe is liable to be exposed to dangerous loads, it is necessary to use pipe of exceptionally high strength, bed it in a cradle of concrete, or use some other material for the sewer. It is evident from what has been said about the manufacture of clay and cement pipe that their tensile strength must be somewhat uncertain and that it is dangerous to copy methods of construction used in laying cast-iron pipe when laying the more brittle sewer pipe.

There is a limit beyond which it is unwise to stress any material. The thicker the shell of vitrified pipe, the more difficult it apparently becomes to burn it uniformly. It will be well, therefore, for the engineer to compare carefully Tables 109, 120 and 121, giving the approximate maximum loads on pipe, and the approximate average breaking loads of pipe tested by uniformly loading the top fourth and supporting the bottom fourth. These tables must be used with care, for the results given in them are averages. If they indicate that a pipe sewer is likely to be laid at a dangerous depth the engineer should not expect too much from the pipe, and he ought to look very carefully after the pipe laying under such conditions.

TABLE 120.—BREAKING LOADS AND PERCENTAGES OF ABSORPTION OF IOWA AND INDIANA VITRIFIED CLAY PIPE (MARSTON AND ANDERSON)

Size, inches	Thickness, inches	Breaking load, lb. per lin. ft.			Absorption percentage
		Maximum	Average	Minimum	
6	0.62–0.75	2,690	1,960	1,690	1.8–3.6
8	0.70–0.80	3,320	1,940	1,460	3.5–4.1
9	0.70–0.80	1,970	1,710	1,430	1.2–1.7
10	0.80–0.88	2,840	1,850	1,210	3.0–4.8
12	0.85–1.10	3,400	2,120	1,370	1.7–4.8
15[1]	1.00–1.30	3,890[2]	2,120	1,220	1.8–4.6
18[1]	1.20–1.50	4,370[2]	2,770	1,570	1.6–4.7
20[1]	1.3–1.8	4,920[2]	2,910	1,720	3.9–4.8
21	1.5–2.0	5,600[2]	4,620	3,030	4.6–5.3
22	1.7–1.7	6,050[2]	5,010	4,530	3.1–4.1
24[1]	1.3–2.1	5,620[2]	3,360	2,050	1.6–4.9
27[1]	2.0–2.4	5,940[2]	4,260	3,080	3.8–5.1
30[1]	2.2–2.7	6,930[2]	5,050	3,530	3.8–4.9
33[1]	2.5–3.0	6,310	4,620	3,970	3.3–4.9
36[1]	2.5–3.0	6,340[2]	4,980	3,900	4.4–5.1

[1] Some of the single-strength pipe developed greater resistance to breaking than some of the double-strength pipe.

[2] Double-strength pipe.

At this time (1913) there is much controversy over absorption tests of clay and cement pipe. Marston and Anderson have reached the conclusion that the maximum permissible absorption by vitrified clay sewer pipe is 4 to 5 per cent., because more absorptive pipe is always unsatisfactory from the viewpoint of strength. The data for cement sewer pipe in their possession are inadequate to warrant any definite conclusion regarding such material. See Tables 120 and 121.

TABLE 121.—TESTS OF BREAKING LOAD AND ABSORPTION OF IOWA CEMENT TILE (MARSTON AND ANDERSON)

Size, inches	Mix	Thickness, inches	Breaking load, lb. per lin. ft Maximum	Average	Minimum	Absorption percentage
4	1:4-1:5	0.35-0.70	1,710	1,130	890	5 9
5	1:3-1:5	0.45-0.75	2,260	1,460	550	6.8-11.3
6	1:3½-1:4	0.45-0.80	2,060	1,190	540	3.9-11.6
7	1:4-1:5	0.55-0.80	2,070	1,290	740	8.8
8	1:3-1:4	0.65-1.35[1]	2,330	1,370	840	8.9-11.3
10	1:3-1:4	0.70-1.45[1]	2,030	1,230	510	7.0- 9.5
12	1:3-1:5	0.75-2.25	5,700	1,510	460	6.2-13.5
14	1:3-1:4	1.05-1.55	2,510	1,380	680	4.4-13.8
16	1:4	1 20-1.70	1,420	1,200	1,000	....
18	1:3-1:4	1.55-2.80	3,860	1,450	600	5.3- 9.5
20	1:3-1:4	1 60-2.20	3,720	1,890	1,110	4.9- 9.6
22	1:3-1:4	1.90-2.45	2,280	1,840	1,480	5.7- 6.6
24	1:3-1:4	1.70-2.50	2,240	1,720	600	5.3- 8.0
26	1:3	2.10-2.70	1,960	1,740	1,360	5.4- 8.2
28	1:3	2.00-2.80	2,240	1,730	1,440	5.5- 8.4
30	1:3	2.50-2.80	1,680	1,570	1,460	5.9- 6.0
32	....	....	2,700	2,110	1,600	....
34	1:3	2.80-3.00	2,070	1,760	1,460	5.2- 7.7
36	1:3-1:4	2.80-3.80	3,230	2,670	1,980	7.0- 9.2

[1] The unusually thick 8- and 10-in. tile were from 30-year old drains taken up in Ames.

The development of the cradle of concrete used at Washington to carry the pipe sewers is shown in Fig. 124. The 1871-79 section had a mortar joint and terra-cotta band and the pipe were without hubs, which has been true of all pipe used down to the present time. When Lieut.-Col. Lansing H. Beach was in charge of the sewerage work there he reported that "the bottom of the sewer, with this pipe, can be made much more even and free from projections due to irregularities of circumference" than with bell pipe. The first section was probably laid many years prior to 1871, according to information furnished by A. E. Phillips, superintendent of the sewer department of the District of Columbia, but that date is the beginning of the public-permitted use of sewers of this type for sanitary drainage. From 1879 to 1888

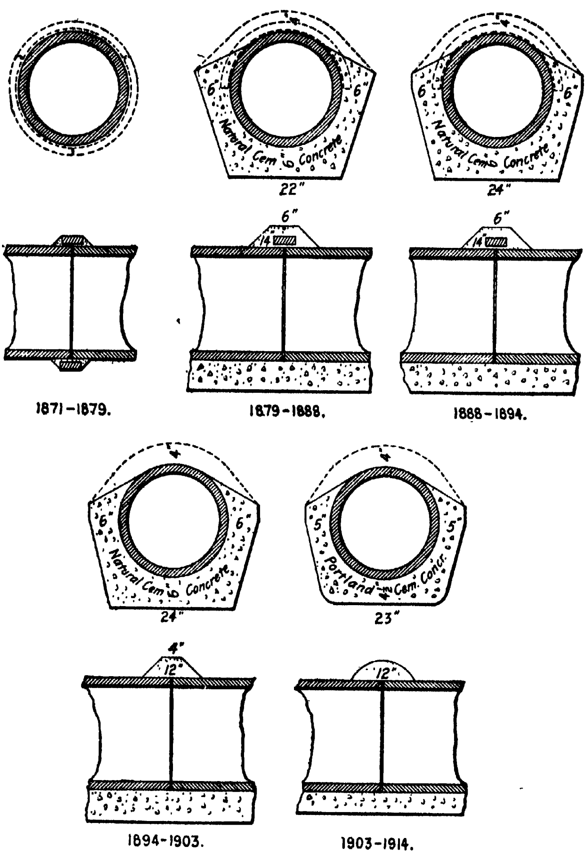

FIG. 124.—Cradle and joint of Washington pipe sewers.

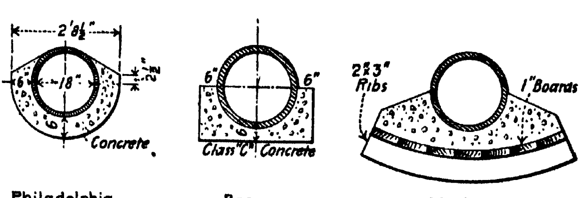

FIG. 125.—Types of cradles.

the pipe rested in a cradle of natural cement concrete 22 in. wide on the bottom and 6 in. thick under the pipe, while the joint was made with a terra-cotta band and a ring of mortar 4 in. thick, 14 in. wide at the pipe and 6 in. wide on top. The 1888–1894 cradle was widened to 24 in. but otherwise it and the joint were unchanged. The 1894–1903 cradle remained unchanged but the terra-cotta band was left out of the joint. The 1903-14 cradle was made of Portland cement concrete and its dimensions were reduced a little, and the joint was given an entirely new cross-section. The concrete envelope was first adopted in 1879, according to Mr. Phillips, as a preventive of root intrusion, by Capt. Hoxie, while engineer commissioner of the District.

Fig. 125 shows three different types of concrete cradles used with bell and spigot pipe.

REINFORCED CONCRETE PIPE

Lock-joint Pipe.—In constructing reinforced concrete sewers in a trench, the practical difficulties lie mainly in making desirable progress, in handling and setting forms, in producing a uniformly dense, hard concrete and in keeping the reinforcement in its proper place while the concrete is deposited about it. The lock-joint pipe was developed by Coleman Meriwether to overcome these difficulties. It consists of a reinforced concrete shell, either circular or egg-shaped, made in 4-ft. lengths; this length has been found economical to handle and materially reduces the number of joints per mile of sewer as compared with the number needed were shorter lengths used. The pipe can be cast with openings to receive standard vitrified clay or cement pipe or slants, where T's or Y's are needed. The usual reinforcement is Triangle Mesh, made by the American Steel & Wire Co., but other materials may be employed. On the larger pipe the shell is reinforced near both the outer and inner surfaces, but in the smaller sizes the inner reinforcement is all that is generally used, usually at a uniform distance from the inside of the pipe. Where the pipe is required to have a flat base instead of a perfectly circular section, the ring of reinforcement is near the inner surface at the top and bottom and near the outer surface at each side. This theoretically desirable position of the reinforcement is practicable where a flat base makes it certain the pipe will always be laid bottom down, but with plain pipe of large size there is some uncertainty about this position being maintained with every length.

The lock joint, Fig. 126, is doubly reinforced. The reinforcement of the shell projects somewhat at each end, so that when the pipe are placed in position the two sets of reinforcement overlap. After a length has been located in its final place in the trench, a metal shield is temporarily

placed inside the pipe, closing the joint, and the latter is filled with thin grout made with cement ground unusually fine. This is usually poured through an opening left in the lip or bell of the shell for this purpose, but sometimes the joints of sewers under 3 ft. in diameter are filled by means of a grout gun, a device for forcing grout into cavities by subjecting it to pressure. The joint made in this way has been repeatedly tested by internal pressure and found to be water tight under all heads to which the sewers were subjected. Circular beams of three lengths of pipe have been made up without special pains in jointing; these have

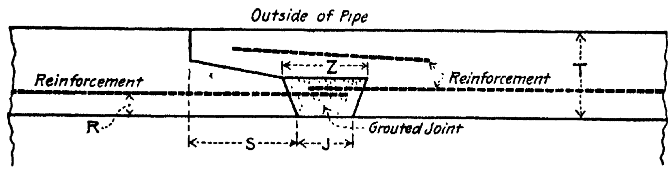

FIG. 126.—The joint of Lock-joint pipe.

been supported near the ends and heavily loaded at the center without causing fracture, showing good locking action of the reinforcement in the joints.

TABLE 122.—STANDARD DIMENSIONS OF LOCK-JOINT PIPE, LENGTH 4 FT. TRIANGLE REINFORCEMENT. (REFERENCES ARE TO FIG. 126)

Diam., in.	*T* in.	*S* in.	*J*, in.	*Z*, in.	*R*, in.	Reinforcement, lb. per sq. ft.	No. of layers of steel	Weight, lb. per ft.
24	3	$1\frac{3}{8}$	$1\frac{5}{8}$	2	1	0.30	Single	250
27	$3\frac{1}{2}$	$1\frac{3}{8}$	$1\frac{5}{8}$	2	1	0.30	Single	350
30	$3\frac{1}{2}$	$1\frac{3}{8}$	$1\frac{5}{8}$	2	1	0.40	Single	380
33	4	$1\frac{3}{8}$	$1\frac{5}{8}$	2	$1\frac{1}{4}$	0.50	Single	480
36	4	$3\frac{1}{2}$	$1\frac{7}{8}$	3	$1\frac{1}{4}$	0.60	Single	520
39	4	$3\frac{1}{2}$	$1\frac{7}{8}$	3	$1\frac{1}{4}$	0.60	Single	580
42	$4\frac{1}{2}$	$3\frac{1}{2}$	$1\frac{7}{8}$	3	$1\frac{1}{2}$	0.60	Single	670
45	$4\frac{1}{2}$	$3\frac{1}{2}$	$1\frac{7}{8}$	3	$1\frac{1}{2}$	0.73	Single	730
48	5	$3\frac{1}{2}$	$1\frac{7}{8}$	3	$1\frac{3}{4}$	0.73–0.83	Single	870
54	$5\frac{1}{2}$	$3\frac{1}{2}$	$1\frac{7}{8}$	3	$1\frac{3}{4}$	1.00	Double	1070
60	6	$3\frac{1}{2}$	$1\frac{7}{8}$	3	6	1.00–1.20	Double	1300
63	6	$3\frac{1}{2}$	$1\frac{7}{8}$	3	6	1.00–1.20	Double	1370
66	$6\frac{1}{2}$	$3\frac{1}{2}$	$1\frac{7}{8}$	3	6	1.00–1.20	Double	1540
72	7	$3\frac{1}{2}$	$1\frac{7}{8}$	3	6	1.20–1.60	Double	1800
78	8	$3\frac{1}{2}$	$1\frac{7}{8}$	3	6	1.60–1.80	Double	2250
84	8	$3\frac{1}{2}$	$1\frac{7}{8}$	3	6	1.80–2.00	Double	2400

Note.—The reinforcement is the minimum used under ordinary circumstances; in a large sewer with very little backfill over it and not likely to be subjected to heavy moving loads, a smaller amount of steel than that stated would be adopted by the Lock-joint Pipe Co., while with deep trenches or heavy extra loads more might be employed.

The manufacture of the pipe is marked by several novel methods developed since the first sewer of this type was laid, which was in Wilmington, Del., in 1908. The concrete is mixed in a small mixer, in which the water is first placed, then the cement, then the sand and finally the 1-in. gravel or broken stone. Experience has convinced the Lock-joint Pipe Co., which controls the Meriwether system, that this results in better mixing for pipe manufacture than the usual procedure with a large mixer. Only a rich mixture, at least 1:2:4, is used, for the company's experience indicates that denser, stronger concrete can be obtained from wet, rich mixtures than from leaner mixtures containing water-proofing compounds but made less carefully. The concrete is usually dumped into a metal pan, where its quality can be readily seen, before it is taken to the molds. If it is poor, the panful is thrown away; but this is rarely necessary when experienced men are employed. The proportions of the mix are fixed by gates placed across the box of the wheelbarrow used in charging the mixer; this method makes it impracticable to alter the proportions except by placing the ingredients in the wrong compartments of the wheelbarrow, which would be quickly detected by the mixer operator.

The molds in which the pipe are made are not sold and are leased only to cities which are putting in sewers by day labor. Where contractors wish to use the pipe, the company manufactures it for them on the spot, for which purpose it maintains its own gangs of experienced men. The company will not allow contractors to make the pipe because of the uncertainty as to what kind of work would be done on a losing contract. The wet mixture is carefully tamped around the reinforcement, which is held firmly in place within the molds. When a mold has been filled, the pipe is steamed for several hours, then the mold is removed, the pipe covered with canvas and steam is again turned on the pipe for several hours. In this way the outside and inside of the pipe are given a finish as smooth as that of hard plaster, except for the presence of occasional small pits.

The concrete thus made is so dense that the company does not advise lining the invert of the sewer with vitrified clay blocks, although it has cast pipe with inverts lined with special tile 1 in. thick and 2 in. wide, interlocked with the concrete. The preference for the concrete over the tile invert is based on examinations of the condition of lock-joint sewers on steep grades after several years of service and on experiments made by Edward S. Rankin, Engineer of Sewers and Drainage of Newark, N. J., (*Proc. Am. Soc. Mun. Imp.*, 1909) which indicated that it was unnecessary to line dense, hard concrete with paving brick or vitrified tile.

While the manufacture of reinforced concrete pipe as compared with the construction of a reinforced concrete sewer in forms in a trench is claimed to be much easier work and of permitting more thorough

inspection, molded pipe has other advantages which are said to have proved helpful in practice. The first is the practicability of making pipe by the method just described in the most severe winters, as demonstrated in Canada and the United States. Another advantage is the narrower trench which can be used with a cast pipe, as was shown convincingly in the narrow streets of Havana, Cuba. A third advantage is the very short trench which need be opened, because as soon as the bottom is reached and prepared, the pipe can be laid and jointed, leaving nothing to interfere with backfilling. With sewers poured in place in the trench, a much longer period must elapse before backfilling. Inasmuch as the joints in the larger sizes of some pipe can be made from the inside, if necessary, it has a special advantage where settlement is feared during backfilling, for the joints need not be poured until after the fill is in place.

In 1913 the company conducted a series of experiments to determine the possibility of laying pipe with lock joints to withstand internal pressures up to about 75 lb. per square inch. The results were so successful that the company decided to take contracts for such pressure conduits, the first closed being a pressure line for the Baltimore water works.

Jackson Pipe.—In the type of pipe made by the Reinforced Concrete Pipe Co., of Jackson, Mich., from five to seven longitudinal reinforcing bars are usually employed and two or three hoops. The wall thickness ranges from 4 in. for 36-in. pipe to 7 in. for the 72-in. size; the usual length is 3 ft. for the medium sizes and 5 ft. for the larger. In the standard type, one end of each pipe is recessed on the inside and the other end has a bevel or taper and a rebate; when a pair of pipe are put together the inner surface is unbroken at the joint and the outer surface has a groove. The longitudinal reinforcing bars project into this groove, and their ends are bent over to form hooks; a band is threaded through these hoops and thus interlocks the longitudinal reinforcement of successive lengths of pipe. When the reinforcement has been coupled in this way, a strip of canvas is placed around the outside of the pipe and held in position by a steel strip. This closes the groove except for an opening about 18 in. long at the top of the pipe, through which thin grout is poured. As soon as the joint has been filled, it is desirable to inspect the interior of the pipe and be sure that there is no indication of any defects, which sometimes are detected in this way.

The actual manufacture of a 72-in. outfall pipe sewer in St. Joseph, Mo., was described in *Eng. Record*, April 28, 1906, as follows:

"In the process of manufacturing the pipe, a bottom plate of cast iron is used, shaped so as to give the flanged or receiving end of the pipe section. The core defining the inside diameter of the section is assembled in four sections of rolled sheet steel on the upper and inner flange of the cast-iron

plate; the longitudinal reinforcing bars are inserted in receiving sockets in the plate and the outer case is then added on the lower and outer flange. The reinforcing bars are held in place at the top by space clips. The circular reinforcing bands are slot-punched, so as to receive and accommodate the longitudinal bars when the bands are put in place, as the process of making is followed. The concrete is shoveled into this form in very small quantities and the tamping is continuous, with the result that there are no layers or creases in the finished pipe. The concrete used in this work was composed of 1 part American Portland cement, 2 parts river sand, and 3 parts crushed limestone; the latter being a mixture of two grades, ranging from pea-size to 1 in. in diameter. The resulting concrete was exceptionally dense."

Occasionally the conditions are such that it is desirable to make the joint as described on only the upper half of the pipe, as it is placed in the trench; the joint for the lower half is made by having the groove on the inside of the pipe, instead of the outside. This enables the pouring of the joint of the lower half of the pipe to be done from the inside, which gives better working conditions under some circumstances.

On the Syracuse, N. Y., intercepting sewers about 11,500 ft. of Jackson pipe had been used up to the close of 1913. This was from 33 to 60 in. in diameter and was made near or at the side of the trench by the Reinforced Concrete Pipe Co., as is the usual custom where this system is employed. The work was done under Glenn D. Holmes, Chief Eng. of the Syracuse Intercepting Sewer Board, whose requirements for reinforcement somewhat exceeded the company's usual practice. They are given in Table 123.

TABLE 123.—REINFORCEMENT IN JACKSON CONCRETE PIPE SEWERS IN SYRACUSE. (GLENN D. HOLMES, CHIEF ENGINEER)

Diameter, inches	Longitudinal bars, inches	Circular bands, inches	Triangle mesh, number	Sheets of metal	Thickness of pipe, inches
33	$\frac{1}{4} \times \frac{1}{2}$	$\frac{1}{8} \times \frac{3}{4}$	3	Single	$3\frac{3}{4}$
42	$\frac{1}{4} \times \frac{1}{2}$	$\frac{1}{8} \times \frac{3}{4}$	1	Single	$4\frac{1}{2}$
48	$\frac{1}{4} \times \frac{3}{4}$	$\frac{1}{8} \times \frac{7}{8}$	1	Single	5
54	$\frac{1}{4} \times \frac{3}{4}$	$\frac{1}{8} \times \frac{7}{8}$	10	Double	$5\frac{1}{2}$

Parmley Pipe.—The reinforced-concrete pipe made by the Parmley & Nethercut Co., of New York and Chicago, is cast vertically in molds with cast-iron bottoms and steel sides, carried on platform cars which are capable of being shaken vertically by a "jolter" or cam device. The cam gives about a hundred 1-in. vertical jolts a minute, which the makers regard as a great help in producing a dense concrete. A plant used in making the pipe in this way is shown in Fig. 127. The pipe are usually made in 4-ft. lengths but 8-ft. lengths are also made; the largest size yet made (1913) is 72-in., although the company has made its designs for an 84-in. size, as stated in Table 124. The pipe are largely used for

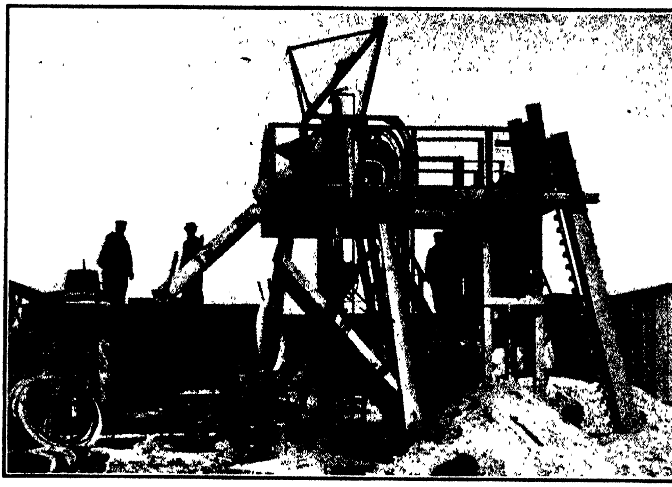

FIG. 127.—Plant used in making Parmley pipe.

culverts by railroad companies. For pressure pipe their most important use has been in 6 miles of 36- and 48-in. aqueduct, under a maximum head of about 60 ft., for the water-works of Fort Worth.

TABLE 124.—DIMENSIONS OF STANDARD PARMLEY PIPE.

Diameter, inches	Width of base, in.	Thickness at crown, in.	Thickness at base, in.	Transverse steel; sq. in per lin. ft.	Steel, lb. per lin. ft.	Weight per lin. ft., lb.
24	8.5	2½	2¾	0.10	4.17	248
27	9.3	2½	2¾	0.11	4.98	275
30	10.1	2¾	3¼	0.13	6.38	330
33	11.0	3	4	0.15	7.27	375
36	11.9	3	4	0.16	8.80	415
39	12.7	3¼	4	0.18	10.51	468
42	13.5	3½	4	0.18	11.22	570
45	14.2	3¾	4	0 21	13.32	648
48	15.1	4	4	0.21	14.13	725
51	16.0	4¼	4¼	0.23	16.34	820
54	17.0	4½	4½	0.24	17.35	915
57	17.8	4¾	4¾	0.24	18.45	1038
60	18.9	5	5	0.24	19.26	1165
66	20.6	5¼	5¼	0.28	23.78	1350
72	22.5	5½	5½	0.34	30.99	1540
78	24.2	5¾	5¾	0.36	34.60	1745
84	25.9	6	6	0.40	41.55	1945

plate; the longitudinal reinforcing bars are inserted in receiving sockets in the plate and the outer case is then added on the lower and outer flange. The reinforcing bars are held in place at the top by space clips. The circular reinforcing bands are slot-punched, so as to receive and accommodate the longitudinal bars when the bands are put in place, as the process of making is followed. The concrete is shoveled into this form in very small quantities and the tamping is continuous, with the result that there are no layers or creases in the finished pipe. The concrete used in this work was composed of 1 part American Portland cement, 2 parts river sand, and 3 parts crushed limestone; the latter being a mixture of two grades, ranging from pea-size to 1 in. in diameter. The resulting concrete was exceptionally dense."

Occasionally the conditions are such that it is desirable to make the joint as described on only the upper half of the pipe, as it is placed in the trench; the joint for the lower half is made by having the groove on the inside of the pipe, instead of the outside. This enables the pouring of the joint of the lower half of the pipe to be done from the inside, which gives better working conditions under some circumstances.

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Diameter, inches	Longitudinal bars, inches	Circular bands, inches	Triangle mesh, number	Sheets of metal	Thickness of pipe, inches
33	$\frac{1}{4} \times \frac{1}{2}$	$\frac{1}{8} \times \frac{3}{4}$	3	Single	$3\frac{3}{4}$
42	$\frac{1}{4} \times \frac{1}{2}$	$\frac{1}{8} \times \frac{3}{4}$	1	Single	$4\frac{1}{2}$
48	$\frac{1}{4} \times \frac{3}{4}$	$\frac{1}{8} \times \frac{7}{8}$	1	Single	5
54	$\frac{1}{4} \times \frac{3}{4}$	$\frac{1}{8} \times \frac{7}{8}$	10	Double	$5\frac{1}{2}$

Parmley Pipe.—The reinforced-concrete pipe made by the Parmley & Nethercut Co., of New York and Chicago, is cast vertically in molds with cast-iron bottoms and steel sides, carried on platform cars which are capable of being shaken vertically by a "jolter" or cam device. The cam gives about a hundred 1-in. vertical jolts a minute, which the makers regard as a great help in producing a dense concrete. A plant used in making the pipe in this way is shown in Fig. 127. The pipe are usually made in 4-ft. lengths but 8-ft. lengths are also made; the largest size yet made (1913) is 72-in., although the company has made its designs for an 84-in. size, as stated in Table 124. The pipe are largely used for

pressures, and a partial vacuum should the pipe be drained while its upper end was closed. The distortion of riveted pipe is a danger by no means remote, and should always receive careful attention.

The steel used in riveted water pipe made under the specifications of engineers who have devoted much study to the subject is usually of the quality demanded by the flange plate specifications of the American Society for Testing Materials. The desire to have special uniformity in strength and good resistance to corrosion is responsible for the requirement that a certain percentage of the top of each ingot from which the plates are rolled, such as 20 per cent. in the case of Allen Hazen's specifications for the Little River water supply of Springfield, Mass., shall be discarded.

Riveting practice in such pipe work is indicated by Hazen's requirements for the Little River supply, just mentioned; these are given in Table 125. These specifications permitted punching the rivet holes

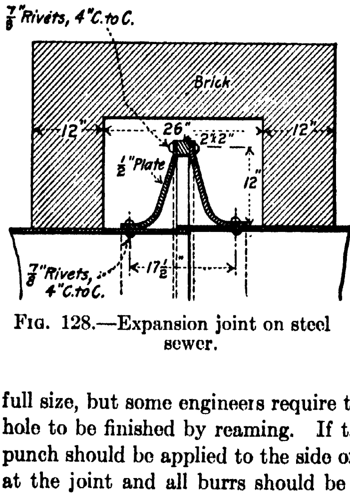

Fig. 128.—Expansion joint on steel sewer.

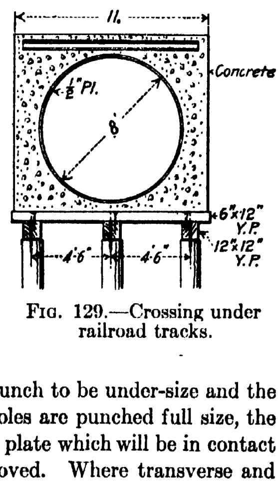

Fig. 129.—Crossing under railroad tracks.

full size, but some engineers require the punch to be under-size and the hole to be finished by reaming. If the holes are punched full size, the punch should be applied to the side of the plate which will be in contact at the joint and all burrs should be removed. Where transverse and longitudinal seams meet, the edges of the plates should be hammered down, and, if possible, two of the rivets of the circular seam should be driven through this part of the joint. The riveting requirements are, as a rule, those of good boiler practice under rigid inspection.

The anchorages for holding the Little River pipe at the gates and the various summits consisted of double blocks of concrete, each 16 ft. long and 13 ft. deep or 13 × 11 ft., about 18 ft. apart and tied to the steel by means of a number of 3-1/2 × 2-1/2-in. angles riveted around the pipe like hoops.

Where steel pipe pass under railway embankments they are either cased in concrete or strengthened with hoops and longitudinal stiffeners of angle iron.

TABLE 125.—RIVETING DIMENSIONS FOR STEEL PIPE (HAZEN)

Thickness of plate, in.	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$
Diameter of rivets, in.	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	$\frac{7}{8}$
Diameter of rivet-holes, in.	$\frac{11}{16}$	$\frac{13}{16}$	$\frac{15}{16}$	$\frac{15}{16}$
Center of rivet to edge of plate, in.	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$
Approximate pitch in all single-riveted seams, in.	1.67	1.9	2.2	2.1
Approximate pitch in double-riveted seams, staggered, in.	2.65	3.1	3.5	3.2
Distance between rows, double riveting, staggered, in.	$1\frac{3}{8}$	$1\frac{1}{2}$	$1\frac{3}{4}$	$1\frac{5}{8}$
Lap of plates, single riveting, in.	$2\frac{1}{4}$	$2\frac{1}{2}$	$2\frac{3}{4}$	3
Lap of plates, double riveting, staggered, in.	$3\frac{5}{8}$	4	$4\frac{1}{2}$	$4\frac{5}{8}$

Note.—These requirements were for 42-in. pipe made in 30-ft. lengths each fourth length being required to pass a shop hydraulic test of 100 lb. for 1/4-in. thickness plate, 150 lb. for 5/16 and 3/8 in. thickness, and 200 lb. for 7/16 in.

Lock-bar steel pipe, in which the longitudinal joint is made by holding the two edges of the sheet together by gripping them in slots in a long bar, instead of by riveting, has been employed for the new 66-in. outfall of the Rochester, N. Y., sewerage system. These pipe are made in this country only by the East Jersey Pipe Co., of New York, and their use has been mainly for water mains.

A grave danger always exists with large steel pipe which may be suddenly emptied, for they are likely to collapse then, flattening out on the ground in such a way as to ruin many of the sheets of which they are composed. This has happened enough times to make it imperative for the engineer in charge of pipe systems containing such steel mains to post warnings against any manipulation of valves or other operations which may cause disaster. Furthermore everyone about the system should be instructed concerning this peculiar danger, in order that any threatening condition will be reported at once to headquarters.

Pipe Coating.—The protection of steel pipe against corrosion has received much attention owing to the pitting of important water mains at Rochester, N. Y., Atlantic City, N. J., and a few other places where unusual care was taken when the pipe were first laid to have them well protected. The definite information on the subject is now (1913) so meager that engineers should keep careful records of the condition of the protective coatings of all riveted pipe just before the backfilling is begun and at all subsequent occasions when the lines are uncovered. This information should also include a careful statement of the nature of the

material surrounding the pipe. It is only in this way that reliable service records can be obtained.[1]

The protective coating adopted by Hazen after long investigation for the Little River works of Springfield, Mass., was made at first from coal-tar pitch distilled until the naphtha was removed, and enough raw linseed oil, free from acid, to make a smooth coat, tough and tenacious when cold and neither brittle nor scaling. Straight-run coal-tar pitch was used; it softened at 60° F. and melted at 100° F., and was a grade in which distillate oils, distilled from it, had a specific gravity of 1.05. The pitch was required to have at least 10 per cent. free carbon, and as much more as was needed to produce the desired qualities in the coating. Subsequently dead oil was substituted for the linseed oil. The material was heated to 350° F. in a tank and the pipe were dipped vertically in it after being brought to the same temperature. This coating is troublesome to apply, and the asphalt pipe dips, which are successfully used between wider temperature limits, are more often employed. Graphite paints have been used to a considerable extent on riveted steel power mains, particularly where they are not buried.

According to testimony by representatives of the American Asphalt & Rubber Co. in the Byerly "blown-oil" litigation, the "Pioneer" pipe dip was composed of about 28 per cent. gilsonite and 72 per cent.

[1] That the preservation of pipes by protective coatings received the earnest attention of engineers many years ago may be easily learned by anyone who will read a report made in 1858 by James P. Kirkwood to the Brooklyn Water Commission in relation to proposals made by various parties to protect cast-iron pipes from corrosion. It is a 62-page pamphlet, of which a copy is in the library of the American Society of Civil Engineers, and reviews the results of advertisements in leading journals of the United States and Europe for proposals for coating the Brooklyn pipes to prevent rusting and tuberculation. Replies were received from England, Scotland, France, Germany and Austria, and some of them showed a knowledge of the effect of different classes of waters on different classes of protective coatings which anticipates the discoveries of a later generation. In fact, in 1839 and 1840 Robert Mollet published in the proceedings of the British Association monographs on the prevention of rusting which it would be well for the enthusiastic commentator on modern technical research to read carefully.

The coating devised by Dr. Angus Smith and now used in a modified form very widely was described by him in 1850 as follows:

"The pipe is made clean, free from rust, and earth which clings to it in coming from the molds. The cleaning is a very important thing, as the success very much depends upon it. The surface is then oiled with linseed oil in order to preserve it until it is ready to be dipped; when the coating is to be made, the pipe is heated in an oven to about 300° F. It should also be managed in such a manner as to prevent soot from settling on it. It is then dipped into a pan of gas pitch and kept in it for some time until it has taken up the pitch as intimately as possible. The pitch should not be too hard, so hard as to be brittle; nor should it be too soft, so as to adhere to anything. When it becomes too hard it may be softened by adding more oil; when the pipes are taken out they are covered with a fine black varnish and look exceedingly well.

"An oven is made to heat the pipes in, and from it they are transferred to the pitch vessel; they are dipped vertically, slowly removed, the liquid running off very clear, leaving a very thin coating. . . . I do not know if you have any distilleries of tar in New York, but, if so, you will readily obtain the proper pitch; we like it distilled till the pitch is about the consistency of wax in our climate. If hard, the mixture of 5 or 6 per cent. linseed oil is a great advantage, or even if not very hard." Letter from Dr. Smith to J. P. Kirkwood.

petroleum residuum, and was prepared by blowing air through the melted materials for 35 to 37 hours. It was one of the so-called mineral rubbers with a high melting point and rather unsusceptible to temperature changes. C. N. Forrest, chief chemist of the New York Testing Laboratory, states that this material will not withstand sunlight and atmospheric conditions for much more than a year; this confirms the experience of engineers who have had pipes coated with these dips exposed for several months along the line of the ditch. Mr. Forrest believes that if the pipe are free from loose scale and are clean it is unnecessary to pickle them before dipping. Both he and Dr. Clifford Richardson are insistent upon keeping the bath at the proper temperature and the proper consistency.

Experience with coatings of wrought iron and steel pipe in California and some neighboring states has been quite different from that in eastern states. The extent to which this is due to differences in soil and water, on the one hand, and to the character of the coating, upon the other, has not been determined. It is probably true, however, that corrosion, pitting and tuberculation of the pipe and blistering of the coating is much less rapid than under conditions in the Eastern United States, except in certain black adobe or highly organic or acid soils and perhaps in some unusually porous soils with very slight covering over the pipe.

The most important lesson to be drawn from these Western experiences seems to be the marked effect of the use of coal tar in pipe coatings in tending to preserve the elasticity of the coating, as indicated by the experience of Hermann Schussler, who enjoys the unique distinction of having guided the engineering development and destinies of one of the largest public service corporations upon the western slope, The Spring Valley Water Co., which supplies the City of San Francisco with water, for a period of substantially fifty consecutive years, and of having built during this time many miles of wrought iron and steel pipe lines for it and for other water and mining corporations in this vicinity. His experience has therefore covered a sufficiently long period of time to be of significance under the conditions there prevailing.

While the early records of his pipe coating methods are not as precise as might be desired, the following description, which has been prepared after conference with him and with employees of The Spring Valley Water Co. and is published by courtesy of its officers, is probably substantially in accord with the facts.

The material used in the coating is composed of a high grade of crude asphaltum, mined at Santa Barbara, and a high grade of domestic coal tar. In the process of refining used, one batch containing about 50 gal. of coal tar is poured into a refining kettle under which a fire has been started, after which 900 lb. of crude asphaltum, previously broken

into chunks from 2 to 4 in. in diameter, is added. As this melts, more asphlatum is added, and the mixture stirred, until a total of about 3,000 lb. is placed in the kettle. A second barrel of coal tar is then added, little by little, to prevent boiling over, as this mixture has been found to give a very tough and tenacious coating on the pipe. This process requires about 8 hours, the boiling taking place at a temperature probably of 300° F. The material is then allowed to boil for about 4 hours without stirring, when the floating dross is skimmed off and the refined asphaltum bailed into a dipping trough, after which the heavy refuse which has settled in the bottom of the kettle is removed and a new charge is put into the kettle. This refuse, consisting largely of sand and gravel, was found upon two recent occasions to average 655 lb. in weight per kettle.

By a slow fire at each end of the dipping trough, the bath is gradually raised to a temperature probably between 360° and 400° F., or if two different troughs are used for the successive immersions of the pipe, the second trough is maintained at a temperatue about 30° less than that of the first, the bath in each case being of sufficient depth to cover the pipe. The consistency of the dip is maintained by the addition of refined asphaltum from the refining kettle and of coal tar, the proper consistency of the coating being tested from time to time by dipping a stick of wood into the bath and, after the coating has cooled, noting its resistance to the point of a knife. In a long run of refining and pipe-dipping, the proportions of the constituent materials of the coating were found to be approximately one 50-gal. barrel of coal tar to 1400 lb. of crude asphaltum.

The cold pipe is immersed in the bath for a period of time sufficient to bring it to the temperature of the dip, which is determined by sliding a bar along the immersed pipe. If the pipe has attained the temperature of the bath, the bar will slide freely upon the metal surface of the pipe; if it has not, it will drag. Pipe 54 in. in diameter, 0.275 in. thick, was found to require immersion for about 25 minutes. The temperature of the bath is maintained by the fires. Before removal from the dipping trough, the bath is vigorously stirred and the pipe is rolled in order to secure a uniform quality of coating. The pipe is then raised above the trough, suspended at an angle of 45° to drain and cool, while the bath is vigorously stirred again. When the coating of the suspended pipe has cooled to a firm and very sticky consistency, the pipe is again immersed in the bath or in the second dipping trough, if such be used, quickly rolled, and after 3 to 5 minutes again removed, suspended at an angle of 45° to drain and cool, and is then lowered to skids coated with dry sand, and removed.

The records are not sufficiently extensive to determine with accuracy the thickness and increase in weight due to the coating. The resulting

thickness of coating is probably about 0.05 in. (or from 0.03 to 0.07 in.), and the increase in weight probably about 0.38 lb. per square foot of surface (coated upon one side only), varying from 10 to 12 per cent. for the thin pipe to about 7 per cent. for 1/4 in. pipe coated upon both sides.

Hundreds of feet of wrought iron pipe coated under Mr. Schussler's direction, of various diameters up to 54 in. which had been in active water-carrying service in the vicinity of San Francisco for various periods of time up to perhaps 47 years, were examined, both inside and out, by the authors. In most cases the coating upon the interior of the pipe was smooth and unbroken, still adhered tenaciously to the pipe, and could be dented readily by the finger-nail and pushed aside by slow hard pressure without cracking, the pipe being clean underneath, showing the mill scale in some cases. Very little corrosion, tuberculation or pitting of the pipe or blistering of its coating was found, the carrying capacity being remarkably well maintained, probably within 10 per cent. of its original amount in most of the pipe, and within 20 per cent. in the oldest and worst case found, as indicated by certain friction loss tests. The exterior coating, while on the whole not in quite such good condition as the interior, was found still generally sound except in those few cases of very limited length in which the pipe traverses soils highly organic or acid in content, such as certain black adobe soils and salt water marshes. The record is a very creditable one.

Further comment on the coatings used on steel pipe on the Pacific Coast is given in the following extract from the final report on the Los Angeles aqueduct:

"A large variety of paints and protective coatings were investigated. It was found that in the use of lead paints the rust scale must be carefully removed, as the paint would not penetrate it but would fleck off, leaving the rust spot beneath. The paint used was a residual hydrocarbon oil, resulting from the manufacture of gas from California asphalt oil. It is different from the eastern coal tars and has the distinct property of penetrating rust and rust scales on the metal. Experience gained from years of its use on sheet steel pipe in this locality demonstrates the long duration of this paint as a protecting medium. During cold weather or on cold plate it becomes necessary to heat and dilute this oil tar with distillate, but with warm conditions dilution is unnecessary. All the steel work on the aqueduct is painted with this material. Its cheapness is another distinct feature, as it costs but $4.00 per barrel of 50 gal. It is applied to the pipe with brushes. There are several trade coal tar paints on the market, but their cost is much greater. One gallon of the paint used will cover about 400 sq. ft. with one coat. The cost of painting with two coats varies from 1/4 cent per square foot under the most favorable conditions, to 1 1/4 cents under the most unfavorable conditions."

In 1913, over 25 miles of large steel pipe were protected by wrapping them with burlap after being dipped in a tank of mineral rubber coating. The process has been developed by the East Jersey Pipe Co. and was described in *Engineering News*, Nov. 27, 1913. The burlap strip is 18 in. wide and is put on in a helical fashion, with an overlap of about 1 in. The company's specifications for the covering read as follows:

"After the pipe has been dipped in the mineral rubber coating and the coating has sufficiently set to prevent flow in the subsequent operations, it shall be wrapped with 10 oz. Calcutta burlap, or equal, which shall be cut into strips 18 in. wide and applied in the following manner: Pipe shall be placed on centers of a wrapping machine where it shall be slowly rotated. The burlap, which shall be carried on the reel of a carriage traveling longitudinally during rotation of pipe, shall be drawn from the reel by the revolving pipe through a tank containing a hot solution of mineral-rubber pipe coating, and shall then be wound spirally on the pipe, the burlap being lapped upon itself to about the width of an inch, the tension of the burlap while winding being sufficient to cause the burlap to lie close and snug on the pipe, but not enough to strain or tear it. The wrapping shall be kept back far enough from the ends of the pipe to leave the rivet holes accessible and not interfere with the making of the field joints. After the pipe is laid, riveted, calked and tested, the field joints are to be wrapped with one wind of the burlap which has been immersed in field coating."

The Institute of Industrial Research, at Washington, made some investigations of the properties of pipe dips in 1913, which Dr. A. S. Cushman, the director, states have given very encouraging results in the case of refined coal tar mixed with linseed oil and a partially soluble basic chromate pigment. Another good class of coatings was made by cutting gilsonite with distilled wood turpentine, to which a very small proportion of a petroleum with a high boiling point was added to do away with the brittleness of the gilsonite.

Bitumastic enamel, which had been used for many years in marine work, was adopted by the Board of Water Supply of New York, for a cast-iron water main laid in salt water. It is expensive, about 7 cents per square foot, and its use on the pipe mentioned was governed by the following specifications:

"After the pipe shall have been inspected . . . and all grease, oil and paint taken off by means of an approved chemical remover, both interior and exterior surfaces shall receive one good coat of bitumastic solution. After delivery, and as short a time as practicable before laying, each pipe shall be given a second good coat of bitumastic solution and immediately thereafter a heavy coat of bitumastic enamel. The solution and the enamel shall each be carefully applied, so as to cover absolutely all the surfaces of the pipe excepting the surfaces above mentioned. Inside the pipe care shall be exercised to make the coating as smooth as possible and to

have such brush marks as may be unavoidable parallel to the axis of the pipe. The enamel shall be of such consistence that it will not scale off when struck a sharp blow with a hardwood instrument nor run when the pipe is exposed to the sun. The consistence of the coating shall be varied as found necessary with the seasonal changes of temperature. The coating as finished shall be free from air bubbles and all other imperfections and nowhere less than $\frac{1}{32}$ in. thick. After each pipe is placed in the line and its joint made, the exposed uncoated portions at the joint shall be coated like the remainder of the pipe and any parts of the coating which may have been injured shall be repaired with enamel or enamel and solution, so as to leave the coating in perfect condition when the pipe is submerged.

The steel pipe used as inverted siphons on the Catskill system of the New York water-works are protected on the outside by at least 6 in. of 1:3:6 concrete and on the inside by 2 in. or more of cement mortar. This protection was adopted after a careful examination of steel pipes in service at the time, 1910-11, and numerous experiments with various coatings. The steel was pickled in dilute sulphuric acid, washed and painted with lime whitewash before it was shipped. The completed pipe line was subjected to a hydrostatic test and the leaks calked, and then, while the pressure was on, the outside concrete was placed. The interior coating was applied in two ways, by the cement gun and by grouting between the pipe and metal-covered wooden molds. The latter method was less costly than the former and gave satisfactory results. In 1913 the coating was found to be cracked but not seriously, according to Alfred D. Flinn; after the pipe had stood full of water for some time and had then been emptied, the cracks closed almost completely.

Mortar lining was used in 1911 on the Weston aqueduct of the Metropolitan water-works on 80-in. steel pipe encased in concrete. The interior of the pipe was cleaned by a sand blast and then given a wash of cement. A Blaw lining was then adjusted by means of set screws through its shell to give a uniform grouting space of 2 in. inside the pipe. This space was filled with 1:2 grout, mixed on a platform moved along on top of the pipe and poured through holes in its crown, other holes being left to allow air to escape. Two men were kept inside the forms beating them with mallets, to drive the air out of the grout.

Flumes.—A modification of steel pipe is used on the outfall sewer of Salt Lake City. In 1911, 2450 ft. of Maginnis semi-circular steel flume was put in. This is 6 ft. 4-1/2 in. in diameter, made of galvanized steel sheets, and is carried every 2-1/2 ft. by a 7/16-in. round rod passing from one end of a crosstie lying on a 4 × 10-in. longitudinal stringer along one side of the trough, down under the trough and up to the other end of the crosstie, which rests on a similar stringer on the other side. These stringers are supported on concrete posts except at a river cross-

ing, where wood piles are used. The Maginnis flume has been employed extensively on irrigation work and its special feature is the joint. This is made by overlapping the plates, a small bead on the lower plate fitting into a groove on the upper plate. A steel rib fits over the joint on the inside and the round carrier rod supports it on the outside; when the bolts on the ends of the carrier are screwed up, the inner rib and the carrier hold the plates together firmly. On the Salt Lake City outlet sewer, a few joints leaked a trifle for two days, but were soon silted up.

Corrugated Pipe.—Corrugated pipe have been occasionally used for sewers, as at Taft and Berkeley, Cal., concerning which lines no information has been obtained by the authors, and at El Paso, Tex. At the last-mentioned place, the pipe line is a temporary one and had to be easily taken to pieces for removal on account of local reasons stated in *Engineering News*, April 17, 1913. It was 24 in. in minimum diameter and the corrugations were 1 in. deep and 2-1/2 in. apart. The pipe was furnished in 30-ft. lengths. Rough gagings of the discharge of the pipe when flowing full indicated that it had a coefficient of rugosity n of 0.0212 to 0.0222 for use in the Kutter formula; these measurements were made at a velocity of flow of about 1-1/2 ft. per second. The head which was required for a flow of 5 cu. ft. per second was 1.49 ft. in 1038 ft. of straight pipe, and 4.37 ft. in 2808 ft. having two right-angle turns with a drop of nearly 3 ft. at one, and a long easy curve at the lower end.

CAST-IRON PIPE

Where internal pressures are heavy or a sewer has to be carried through a wall where it is rigidly held, cast-iron pipe are usually laid until the sewer has passed well outside the danger zone. The relations of bursting pressure and thickness and the standard lists of cast-iron pipe are taken up in detail at an earlier point in this chapter. It is important to keep in mind, however, that in pipe of large diameter with thin shells, such as are likely to be used for sewers, external pressures may be more dangerous than internal pressures. For this reason it is sometimes desirable to surround the pipe with concrete, which in the case of pipe over 20 in. in diameter, crossing under streams, serves also to weight down the pipe and keep it from rising when empty. Smaller sizes are usually too heavy proportionally to float.

While most of the sewerage uses of cast-iron pipe are for inverted siphons and outfalls, they are occasionally employed in crossing under railroad tracks, aqueducts and like structures which it is desirable to keep protected against every kind of structural danger. For example, a cast-iron sewer used at Tompkinsville, Staten Island, N. Y., is shown in Fig. 130. This drains a 47-acre hillside and is likely at times to be

under a head of 13 ft. In a distance of 535 ft. it crosses under two main passenger tracks and 24 switch tracks, at depths of 2 or 3 ft. This yard is crowded with cars so that the live load on the sewer is likely to be heavy, and in addition the yard is located on a rather soft fill, which made it necessary to use piling, 12 to 36 ft. long, in order to support the structure on a firm stratum. The surface water collecting on the yard is carried off in the vitrified pipe drain beside the sewer in the illustration.

Where cast-iron pipe lines cross a stream on or below the bed of the river, it is important to protect them from injury of every sort, including damage during floods. In some cases this can be attained by merely laying the pipe in a trench, in other cases it is desirable to surround them with concrete and in some cases, where the pipes are supported on piling, they should be held in a frame over every pile bent, and by pine blocks

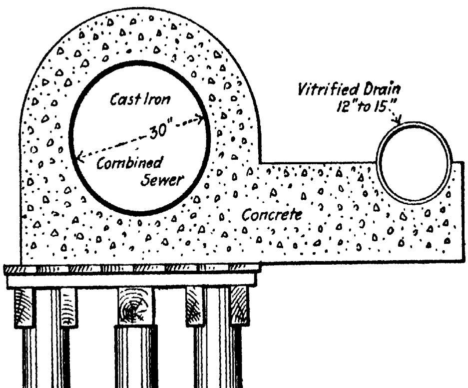

FIG. 130.—Cast-iron sewer under railroad yard.

below them on the cap, so that neither vertical nor horizontal motion of any sort is possible. For example, in the crossing of the Don River at Toronto the sewage is carried in two pipe lines, one 3–1/2 and the other 5 1/2 ft. in diameter, which are spaced 6 ft. apart on centers on pile bents. The water at this crossing is 5 ft. deep and the tops of the pipe are 10 ft. below the river bed. There are two pile bents to each 8 ft. length of pipe, and over each bent is a frame of 3 × 10-in. plank, which holds the pipe securely on the blocks bolted to the cap of the piles.

The cast-iron pipe lines, 42 and 48 in. in diameter, which form the outfalls of the sewerage system of Waterloo, England, are supported on hollow cast-iron jet piles 8 ft. long. In a paper published by the Institution of Civil Engineers, Mr. Ben Howorth describes the process of

ing, where wood piles are used. The Maginnis flume has been employed extensively on irrigation work and its special feature is the joint. This is made by overlapping the plates, a small bead on the lower plate fitting into a groove on the upper plate. A steel rib fits over the joint on the inside and the round carrier rod supports it on the outside; when the bolts on the ends of the carrier are screwed up, the inner rib and the carrier hold the plates together firmly. On the Salt Lake City outlet sewer, a few joints leaked a trifle for two days, but were soon silted up.

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the staves due to the pressure of the water. A paper on this subject was contributed to *Trans. Am. Soc. C. E.*, June, 1899, by A. L. Adams. As a result of his investigations he recommended the adoption of the dimensions given in Table 126, the spacing of the bands to be determined in each case by the engineer to meet the pressures the pipe must carry.

The band spacing on the wood-stave pipe of the Denver Union Water Co. is determined by the formula

$$N = 260DH/AS$$

where N = number of bands per 100 ft. of pipe,
D = inside diameter of pipe in inches,
H = head of water in feet,
A = sectional area of band in square inches,
S = safe tensile strength of steel in pounds per square inch.

The company uses both 12,000 and 15,000 lb. as S, and if these values are substituted in the formula it takes the following forms for bands of different cross-sections:

Diam. bands, in.	3/8	1/2	5/8	3/4
S = 12,000 lb.	$DH/5$	$DH/9$	$DH/14.1$	$DH/20.4$
S = 15,000 lb.	$DH/6.4$	$DH/11.3$	$DH/18$	$DH/25.5$

The spacing between centers of the bands in inches, f, adopted by D. L. Henny, is determined by the formula

$$f = \frac{S}{P(R + \frac{3}{2}t)}$$

where S is as given above, P is the water pressure in pounds per square inch, R is the internal radius in inches, and t is the thickness of the staves in inches. In *Trans. Am. Soc. C. E.*, vol. xli, p. 72, he stated that 12 in. was the maximum spacing he used with staves 2 in. thick in small pipe, and this was reduced to 11 and 10 in. as the diameter of the pipe increased. With 1½-in. lumber the maximum spacing was 10 in. In using the formula it is necessary to make sure that the pressure of the bands on the staves does not exceed a safe amount, which Mr. Adams gave as 800 lb. per square inch of band contact, whereas Mr. Henny preferred to adopt a sliding scale of values, ranging from 140 lb. per lineal inch of band with 3/8-in. rods, to 262 lb. with 7/8-in. rods. This is equivalent to 747 to 600 lb. per square inch for the same range of sizes. The usual practice is to estimate this pressure by means of the formula, $e = S/(R + t)$, where e is the desired unit pressure and the other letters represent quantities as previously stated.

Owing to the necessity of keeping the wood saturated to the outside of the pipe, in order to prevent decay, it is a positive disadvantage to

use staves with a greater thickness than is needed to withstand the service conditions.

The pipe used in the western section of the country are mainly of Oregon fir or redwood staves, while eastern specifications have permitted white pine, yellow pine and white cedar. In any case, it is desirable to employ only absolutely clear stock of the highest quality, for if wood containing sap and pitch is introduced into the line, experience shows that an element of dangerous uncertainty will be admitted. Wood of close, even texture is preferable to that of a more coarse character. The staves are usually milled to brass templates, either made or checked by the engineer, and the use of beaded edges, formerly much favored, has few advocates now except for pipe to be used under small heads, which do not require heavy cinching of the bands. Slight beads along one edge of the stave probably help to make such a pipe tight.

TABLE 126.—DIMENSIONS OF WOOD-STAVE PIPE DETAILS (ADAMS)

Diam. of pipe, inches	Stock size of staves, inches	Finished thickness of staves, inches	Best size bands, inches	Band cross-section
10	$1\frac{1}{2} \times 4$	$1\frac{1}{16}$	$\frac{5}{16} \times \frac{7}{16}$	Elliptical
12	$1\frac{1}{2} \times 4$	$1\frac{1}{8}$	$\frac{5}{16} \times \frac{7}{16}$	Elliptical
14	$1\frac{1}{2} \times 4$	$1\frac{3}{16}$	$\frac{5}{16} \times \frac{7}{16}$	Elliptical
16	2×6	$1\frac{7}{32}$	$\frac{5}{16} \times \frac{7}{16}$	Elliptical
18	2×6	$1\frac{3}{8}$	$\frac{5}{16} \times \frac{7}{16}$	Elliptical
20	2×6	$1\frac{3}{8}$	$\frac{5}{16} \times \frac{7}{16}$	Elliptical
22	2×6	$1\frac{3}{8}$	$\frac{3}{8}$	Round
24	2×6	$1\frac{3}{8}$	$\frac{3}{8}$	Round
27	2×6	$1\frac{7}{16}$	$\frac{3}{8}$	Round
30	2×6	$1\frac{1}{2}$	$\frac{1}{2}$	Round
36	2×6	$1\frac{9}{16}$	$\frac{1}{2}$	Round
42	2×6	$1\frac{5}{8}$	$\frac{1}{2}$	Round
48	2×6	$1\frac{11}{16}$	$\frac{1}{2}$	Round
54	$2\frac{1}{2} \times 8$	$2\frac{1}{8}$	$\frac{5}{8}$	Round
60	3×8	$2\frac{1}{2}$	$\frac{5}{8}$	Round
66	3×8	$2\frac{9}{16}$	$\frac{3}{4}$	Round
72	3×8	$2\frac{5}{8}$	$\frac{3}{4}$	Round

The bands used to cinch the staves into a pipe, being the weakest feature according to experience, should be made of good wrought iron and covered with a durable paint or heavy varnish of the pipe-dip nature. One end should be upset and have a thread rolled in it, and the other is attached in various ways to the coupling shoe by which the rod is made into a band. The malleable cast-iron shoe evolved from the early experience with the wood-stave pipe of the Denver Union Water Co. and extensively used by the Excelsior Wooden Pipe Co., is perhaps the favorite type on irrigation and water-supply works.

The wood-stave outfall sewer built in 1903 at New London, Conn., Fig. 131, was typical of such work done in that section of the country at that time. The outlet was submerged 9 ft. below high tide, at the top of a sharp slope, and was located 900 ft. from shore. The 18-in. outfall was 1600 ft. long and terminated at a small catch-basin, from which a 10-in. sewer ran inland for a distance of 1600 ft. on a 0.1 per cent. grade. The entire sewer was submerged at mean high tide and where it passed through the catch basin it was 2–1/2 ft. below high tide, so that it was entirely emptied only twice in 24 hours, at low water, when it was cleaned by a discharge from a large flush tank at the upper end.

This sewer was laid in a trench excavated in the mud and was covered to a depth of 2 to 12 ft. with mud. During this work a wood-stave

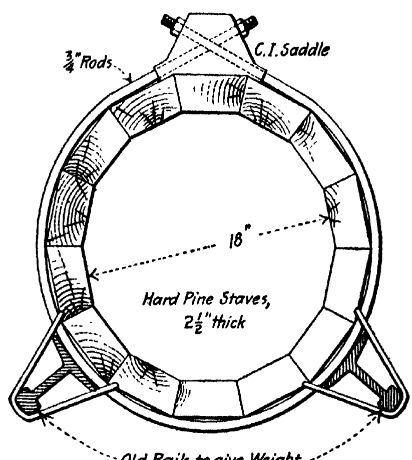


FIG. 131.—New London outfall sewer.

outfall sewer, built in 1893 of green cypress staves and damaged later by dredging, was uncovered in a number of places and found to be in good condition, not even the hoops showing signs of deterioration. Six of these wood-stave outfalls have been laid in New London under the direction of W. H. Richards, engineer and superintendent of the Water and Sewer Department. The larger ones have given no trouble, but the smaller ones, receiving the sewage from very small areas which consequently has a very low velocity, have become obstructed by grease at times and in a few cases the hoops have been badly corroded.

A 14-in. wood-stave pipe was built for the outfall of the Ithaca, N. Y., sewerage system, in 1895. The pipe had eleven 3-in. hemlock staves and

was much like that at New London, Fig. 131, being weighted with old 60-lb. steel rails in the same fashion. The hoops were 3/4-in. galvanized iron bands spaced 4 ft. apart. To construct this outfall, a platform 20 ft. wide was built out from the shore line, for a distance of about 100 ft. along the line of the outfall. Rollers were placed along this platform every 8 ft., and the stave pipe was put together on these rollers, a rough form of laths being used as a sort of center in this work. When about 100 ft. of pipe had been put together, it was rolled off the end of the platform into the water, after empty oil barrels had been attached to it. When about a fourth of the outfall had been built in this way it was towed into place alongside guide piles. The rails were heavy enough to sink the pipe when the oil barrels were detached. The four sections of sewer were joined by oak staves 16 ft. long, encircling the pipe and nailed to it, with special bands put around the entire joint. This outfall was abandoned after about 5 years of service, on account of a change in the method of sewage disposal. During its service, the only faults observed in it, according to Henry L. Stewart, assistant superintendent of public works, were a leakage between some of the staves, due to spacing the hoops too far apart, and a tendency for the hoops to break when they were bent into small circles. An inspection of the staves in 1913 showed them to be apparently sound.

Wood-stave pipe have been extensively used for many years in outfall sewers carried on timber piers. Their lightness and flexibility are particularly desirable in such situations. Usually the pipe are carried by the same piles which support the deck of the bridge, but occasionally semi-independent bents are driven, as for the 3-1/2 ft. sewer on Pier 19 in Philadelphia (*Eng. Record*, Feb. 10, 1912). Here pairs of piles on 6-ft. centers, the pairs spaced 10 ft. apart, were driven along the axis of the pier. Each pair of piles was capped with two 6 × 12-in. clamps, the pile tops being notched to receive them. These caps were long enough to be fastened on each side of the sewer to other piles than those driven primarily for the sewer. They carried longitudinal 12 × 12-in. stringers, to which the pipe were strapped at alternate intervals of 7½ and 2½ ft.; the straps were 7/16 × 3 in. with round ends which were carried down through the stringers and held by nuts and washers. The pipe were also held by chocks placed between them and the stringers. Five-inch staves were used in their construction and these were held by 7/16 × 3-in. semi-circular straps bolted together through lugs on their ends to form hoops. The straps over the piles were made 1 in. wider.

The largest wood-stave sewers in the United States at this time (1913) are the twin 60-in. underwater discharge conduits 2500 ft. long through which the effluent from the Baltimore sewage treatment works is delivered into the Black River. These sewers were designed by Calvin W. Hendrick, Chief Eng. of the Baltimore Sewerage Commission.

The contractors were permitted to use cypress, redwood, fir, longleaf yellow pine or other wood satisfactory to the engineer, and the staves were not allowed to be less than 12 ft. long and 2 in. thick nor more than 8 in. wide. The bands were 3/4-in. round bars on 18-in. centers or 5/8 in. bars on a smaller spacing. The twin pipe were laid on two-pile bents 25 ft. apart; the piles were driven until their points were at least 10 ft. below the final location of the bottom of the pipe. Between the two rows of piles, a trench was excavated deep enough to allow the pipe to be lowered until their tops were 2 ft. below the river bottom, where they were held by 3 × 12-in. caps.

CHAPTER XI

THE DESIGN OF MASONRY SEWERS

The majority of the masonry sewers constructed in this country have been of circular cross-section, although in some old systems many sewers constructed with an oval or egg-shaped section are to be found. Since about 1900 a number of other sections have come into use and some of them have found quite general favor. In the following paragraphs the principal types are described and some of their chief advantages and disadvantages discussed.

Circular Section.—The circular section has been used more often than any other. It encloses a given area with the least perimeter and on that account gives the greatest velocity when flowing half-full or full. Under ordinary conditions circular sections are economical in the amount of masonry required, although in flat bottom trenches or under conditions requiring special foundations, such as piles or timber platforms, additional masonry is required to support the arch. In the combined system, where the dry-weather flow of sewage is very small in comparison to the storm-water flow, the velocity for the low flows is greatly reduced in the circular section and on that account this section may not be as advantageous as the egg-shaped section.

For sewers under 5 ft. in diameter the circular or egg-shaped forms are usually employed in preference to other types.

Egg-shaped Section.—In combined sewers where the dry-weather flow of sewage is small compared with the capacity of the sewer required for storm water, or in sanitary sewers for a district where the present population is but a small proportion of the ultimate development, the ideal sewer section is one in which the hydraulic radius remains constant as the depth of flow decreases. It is impracticable to obtain the ideal, but the egg-shaped or oval section comes nearer to it than any other thus far devised.

In some cases the attempt has been made to design an egg-shaped section to meet special conditions, such as limited head room, or to proportion the radii of the oval to provide for special variations between the normal and maximum flows. This has led to some forms which have found little favor in this country, although used extensively abroad.

The standard egg-shaped section shown in Fig. 132a, was designed in England by John Phillips about 1846, and has been used considerably

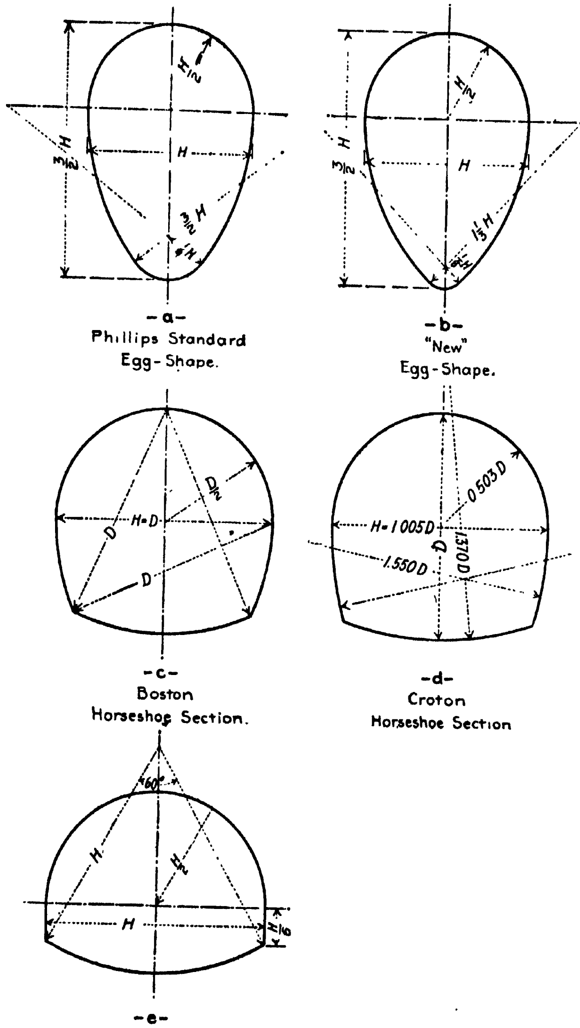


FIG. 132.—Typical cross-sections of sewers.

since that date without modification. He also designed a "new" egg-shaped section, shown in Fig. 132b, for use where the normal flow is extremely small compared with the maximum, but this has not been used so largely. The advantage of the egg-shaped sewer is that for small flows the depth is greater and the velocity somewhat higher than in a circular sewer of equivalent capacity. This is well illustrated by Table 127, showing the comparative depths and velocities in a 6-ft.

TABLE 127.—COMPARATIVE VELOCITIES IN CIRCULAR AND EGG-SHAPE SEWERS BY KUTTER'S FORMULA WITH $n = 0.013$

Circular sewer 6 ft. diam, $S = 0.00025$, $Q = 67.4$ c.f.s.

Egg-shape sewer of equivalent capacity, $S = 0.00025$, 5 ft $\times$ 7 ft. 6 in., $Q = 66.9$ c.f.s.

Quantity flowing in sewer, cu. ft. per sec.	Ratio of quantity flowing to capacity of circular sewer full	Depth of flow in feet		Velocity, ft. per sec.	
		Circular	Egg-shape	Circular	Egg-shape
0.03	0.0005	0.11	0.14	0.24	0.30
0.34	0 005	0.32	0.40	0.57	0.65
0.67	0.010	0.44	0.56	0.73	0.81
1.01	0.015	0.53	0.68	0.84	0.93
1.35	0.020	0.65	0.78	0.91	1.00
1.7	0.025	0.72	0.90	1.00	1.05
3.4	0.05	0.96	1.28	1.22	1.28
6.7	0.10	1.29	1.76	1.48	1.54
13.5	0.20	1.82	2.51	1.84	1.85
20.2	0.30	2.27	3.09	2.08	2.05
27.0	0.40	2.64	3.61	2.25	2.21
33.7	0.50	3.00	4.09	2.39	2.32
40.4	0.60	3.34	4.56	2.50	2.41
47.2	0.70	3.69	5.00	2.59	2.49
53.9	0.80	4.05	5.40	2.66	2.55
60.7	0.90	4.45	5.85	2.72	2.60
67.4	1.00	4.92	6.36	2.74	2.63
72.8	1.08	5.64	7.05	2.64	2.54
67.4	1.0	6.00	7.50	2.39	2.33

circular sewer and in an equivalent egg-shaped sewer 5 ft. by 7 ft. 6 in., for various equal flows in each type. For the small flows amounting to two-tenths of the total capacity of the sewer or less, the velocity in the egg-shaped sewer is somewhat greater than in the circular. While this difference may not be of practical benefit of itself, it is of value when considered with the increase in depth, and taken together, these differences make the egg-shaped sewer more desirable, where the flow in a large sewer is at times very small. The depth of flow in the egg-shaped sewer is always greater than in the circular sewer for equal quantities, and for the small flows this increase in depth produces better flotation for the solid matter and consequently better actual velocity than where the depth of flow is less and the actual velocity is con-

sequently less than that computed, because of the obstructions caused by solid matter. Whether or not the advantage in greater velocity and depth of flow is sufficient to offset the disadvantages must be determined for each particular case.

For sewers 6 ft. in diameter and over, it is doubtful if the egg-shaped section is sufficiently economical. As may be seen by the example shown in Table 127, the circular sewer has a vertical height of 6 ft., while the egg-shaped sewer requires a height of 7 ft. 6 in. On the other hand, the horizontal diameter is decreased from 6 ft. in the circular, to 5 ft. in the egg-shaped, which makes it possible to construct the sewer in a narrower trench. In deep trenches there will be a saving in total excavation by using the egg-shaped sewer, due to the decrease in width of trench, which may more than offset the small increase in depth.

The egg-shaped section has the disadvantages, however, of being less stable, more liable to crack, requiring more masonry, and in general being more difficult to construct. In very stiff soil or in rock it is sometimes possible to excavate the bottom of the trench to conform to the shape of the invert of the sewer, but in general, in yielding soil or where foundations are poor, requiring piles or timber platforms, the egg-shaped section requires considerable masonry backing under the haunches to support the arch, even more than in the case of the circular sewer. For this reason the egg-shaped section will be found in many cases much more expensive than the circular type and far more expensive than some of the other types which are discussed further on.

Catenary Section.—This section was used extensively on the Massachusetts North Metropolitan sewerage system, under the direction of Howard A. Carson. Its principal advantage is in the fact that it conforms so nearly in shape to the available space inside the wooden timbering in earth tunnels, as may be seen in Fig. 157. The section is strong in that the line of resistance keeps well within the arch section, has fairly good hydraulic properties and carries the center of gravity of the wetted area much lower with respect to the height than the circular section. This last fact may be of some advantage in locating lateral connections at a lower elevation, or in raising the invert of the main sewer. This, of course, contemplates the possible operation of the lateral sewers under a head at times when the main sewer is running full. There are cases where this may be a practical scheme, but in general it should be avoided. It is of material advantage, however, where the allowable difference in water level is small. A larger quantity can be carried for a given increase in depth than is the case with the circular sewer. The catenary section has been but little used of late years.

Gothic Section.—This section, closely resembling the circular in shape and advantages, was also used to some extent on the North Metro-

politan sewerage system in Massachusetts; see Fig. 135, page 393. The horizontal diameter is about 17 per cent. less and the vertical diameter about 8 per cent. more than the diameter of the equivalent circle, and on that account it requires less width of trench than the circular section. Its greater height may be disadvantageous except under special conditions, because of the increased quantity of excavation required when the sewers are located on the basis of the crown grade. As may be expected, the hydraulic properties are not far different from those of the circular section. The Gothic or pointed arch is somewhat stronger than the semi-circular. This section is not in general use at the present time, although it has advantages for special cases.

Basket-handle Section.—This section, Fig. 159, was developed by Howard A. Carson on the Massachusetts North Metropolitan sewer work, and has been used to a large extent on that system, and also to some extent in other places. It is so nearly a horse-shoe type that it is hard to draw a definite line between the two. Concerning this standard section, Fig. 159b, Mr. Carson states in his Third Annual Report to the Metropolitan Sewerage Commission, for the year ending Sept. 30, 1891, the following:

"The horizontal diameter is about 6 per cent. less than the vertical. The arch is slightly pointed and the invert is flatter than a semi-circle. In this shape the area, perimeter and the theoretical velocity when flowing more than one-sixth full differ but little from the corresponding elements in a circle having the same height. In actual construction, under the conditions that usually obtain on our work, this shape is more stable when entirely completed, than a circular shape. It requires more care, however, to prevent injury to the invert, while the latter is being constructed."

In general the basket-handle section has about the same advantages and disadvantages as the horse-shoe type, described next, into which it merges so that it is difficult to determine whether some sections should be classed as basket-handle or horse-shoe. The invert with the large radius curve and rounded corners between the side walls and invert, may be of some advantage for strength, but the difference is so slight and the difficulties of construction so much greater than the form usually employed in the horse-shoe type that it is not now in general favor. There may also be some slight advantage in having the Gothic arch because of greater strength and somewhat greater ease in removing collapsible arch forms.

Horse-shoe Section.—For large sewers it is probable that next to the circular section this has been more generally used than any other. Many horse-shoe sections have been developed to meet varying conditions, a few of which are shown in Figs. 160, 161 and 162. Above the springing line the horse-shoe section has a semi-circular arch, while the

side walls below the springing line are vertical or incline inward, sometimes with a plain and sometimes with a curved surface. The surface of the invert varies in section from a horizontal line to a circular or parabolic arc, or other design calculated to concentrate the low flows near the center of the invert.

One great advantage of this type of sewer is that it conforms to the shape of the bottom of the trench as usually excavated, and on that account does not require the large increase in masonry backing to sustain the arch, which is needed with circular and egg-shaped sewers built in anything but the firmest of soil or rock. Another advantage is that for a given width or horizontal diameter the sewer may be designed with less height as a horse-shoe and still have the same carrying capacity as the circular. Where the depth of the sewer below the surface is controlled by the grade at the crown, there would be a consequent saving in excavation because of the decrease in depth. Where only a restricted amount of head room is available, the wide horse-shoe type can often be used to advantage. In yielding soil where it is necessary to spread the foundations, the horse-shoe type can be used in an economical manner, because of the saving of masonry in the invert. The limit of the horseshoe section along this line is the semi-circular section, in reality a horse-shoe section without side walls.

The chief disadvantage of the horse-shoe section is that unless the side walls are made heavy the stability of the arch must depend to some extent on the ability of the earth backfilling to resist the lateral thrust of the arch transmitted to the side walls near the springing line. The effect of the side walls is to increase greatly the bending moment at the crown and center of invert, especially the latter. If the sewer is constructed of monolithic reinforced concrete, with continuous reinforcing bars from the center of the invert to the crown of the sewer, the bending moment developed at the crown and invert center will be very severe and also the bending moment at the springing line.

If the horse-shoe section is constructed in rock cut, so that the invert or base of the side walls can be built directly on the rock the conditions will be greatly altered and the line of thrust will stay within the section much more readily than if the sewer is constructed in compressible soil, with the whole structure acting as an elastic arch from the center of the invert to the crown. In the case of brick arches this necessitates the construction of comparatively heavy side walls or abutments. The use of reinforcing metal in concrete has helped to remedy this trouble, but even with heavily reinforced sections cases are known where the arch cracked on the inside at the crown and on the outside at the quarter points or down toward the springing line. While this did not produce failure, it was objectionable from the point of view of leakage and rusting of the steel reinforcing metal. It is hard to conceive

of the passive resistance of the earth, especially in newly backfilled trenches, being brought into action without some movement of the concrete to compact the particles of earth next to the masonry. The effect of such movement on the stability of brick arches is well illustrated in a paper by Alphonse Fteley on "Stability of Brick Conduits," *Jour. Assoc. Engr. Soc.*, Feb., 1883.

Semi-elliptical Section.—The arch of this section is either a true semi-ellipse or is made up of three circular arcs thus approximating the semi-ellipse. As the center of gravity of the wetted area is lower down with respect to the crown than in the circular sewer, the normal flow line will be much lower which, as mentioned in the discussion of the catenary section, may be of considerable advantage. (See Fig. 163, page 441.)

The chief advantage of this type of sewer is that the shape of the arch more nearly coincides with the line of resistance of the arch under actual working conditions than is the case with other sections. Because of this the arch section can be made relatively thin and still keep the stresses in the masonry within allowable limits. The section is dependent to only a small extent on the lateral pressure of the earth to prevent failure, and does not depend upon the passive pressure or natural resistance of the earth filling on the sides, as is the case, very often, with the horse-shoe type. The fact that the arch is of thin section and goes nearly to the invert line, makes it more necessary to design and construct the invert of the semi-elliptical type, so as to distribute the pressure over a sufficiently large area.

This section depends to a larger extent on the stability of the invert than is the case with the sections previously mentioned. Where the semi-elliptical section is constructed in compressible soil and the structure is built monolithic, with reinforcing bars running continuously from the center of the invert to the crown of the sewer, there will be a large bending moment at the center of the invert. Under such foundation conditions, the invert should be made as thick as the arch at the springing line and should be heavily reinforced to withstand the stresses. Unless this is done cracks are likely to occur at the center of the invert.

As in the horse-shoe type, the invert of the semi-elliptical section readily conforms to the bottom of the trench excavation, and for that reason the quantity of masonry below the springing line is not excessive.

This section is not as advantageous for low flows as the circular, because of the wide and shallow invert in which there is a very low velocity. However, for sewers where the quantity to be carried is not subject to wide variations and the normal flow is as much as one-third of the total capacity of the sewer, this disadvantage may be neglected. The hydraulic properties of the semi-elliptical section are very good in general, which, with the very desirable structural features, make this type one of the best for sewers over 6 ft. in diameter.

Parabolic or Delta Sections.—This type of sewer was designed and built by W. B. Fuller at Duluth, Minn., in 1888 (*Eng. News*, Oct. 25, 1890) and later by James H. Fuertes in connection with the design of the sewerage system for Santos, Brazil, briefly described in *Eng. Record*, March 17, 1894. In 1902 Fuertes designed a similar section for Harrisburg, Pa., shown in Fig. 166a.

The sewer section is nearly triangular in shape, the arch being a parabola and the invert a short circular arc with side slopes of about 3 horizontal to 1 vertical. The section shown has a somewhat larger carrying capacity than that of a circular section of the same height. It is both economical and strong and has the added advantage that the normal flow line is lower than in the circular section. This is especially valuable in districts where the available fall is limited, as in cities where the effect of tide water requires the sewers to be built in shallow cut. The sloping invert is well adapted for low flows. In this section, as in the semi-elliptical type, the shape of the arch very nearly coincides with the line of arch resistance, which results in a very strong section. It has the disadvantage as compared with the semi-elliptical type, however, of requiring a wider space for equal capacity and height, because of the pointed arch. For locations where there is but little depth of excavation, as in crossing low land, the section has a further advantage because the wide space makes it possible to construct the foundation to better advantage and the greater carrying capacity below the springing line makes it possible to build a section of less height than in the case of the circular sewer, and where the sewer has to be covered by an embankment this involves a smaller quantity of earth work.

Elliptical Section.—A few sewers have been constructed in this country with a true elliptical section, see Fig. 158, some with the longer axis vertical and others with it horizontal. This section is unlike the semi-elliptical type in that both above and below the springing line, it is an approximation to a portion of an ellipse. Unless the excavation is made in very firm soil, there will be additional masonry backing required under the haunches to support the arch, as in the case of the egg-shaped section, and this is usually an objection from the point of view of economy in masonry. In general, this shape is difficult to construct and because there are so few points to commend the section, it has not been brought into general use.

U-shape Section.—In cases where the width of trench is limited and sufficient head room is available to build a sewer whose vertical diameter is materially greater than the horizontal, the U-shape section has some advantages. See Fig. 168e and f. The hydraulic properties of the section of Fig. 168e are fairly good until it becomes filled, when the hydraulic mean radius becomes materially reduced due to the addition of the width of the slab roof to the wetted perimeter. The invert is well adapted to maintain good velocities for low flows. It also has the

advantage, because of the pointed shape, of offering a little less difficulty to the withdrawal of forms than the circular invert. In proportion to its area it requires considerable masonry and on that account is not economical for large sewers, but for sewers in the vicinity of 3 ft. in diameter, it doubtless has advantages for special conditions.

Rectangular Section.—This type has been used for many years where the head room or side room in the trench was limited, but more recently the rectangular section has been used for main lines because of the simplified form work, easy construction, economy of space in the trench, and economy of masonry, and also because of its excellent hydraulic properties at all depths previous to the flat top becoming wet. As may be seen from the diagram of hydraulic elements, Fig. 144, the velocity and discharge are relatively large just before the flat top is wet, but decrease very suddenly as soon as the wetted perimeter is increased and the hydraulic radius decreased by the wetting of the top. On this account it is customary in designing rectangular sections to allow an air space above the maximum flow line of from 3 to 12 in., depending upon the size of the sewer and the amount of head room available.

Where the trench is in deep rock cut, this type can be used to great advantage, as is pointed out by W. W. Horner in *Engineering News*, Sept. 5, 1912. With a narrow, high section, the width of excavation can be materially reduced, often more than enough to offset the increase in depth of trench required. The hydraulic properties of the section become less favorable as the ratio of the height to the width increases; Mr. Horner found the economical ratio to be between 1.5 and 2. A section of this type is shown in Fig. 165b. The usual form of rectangular section has a greater width than height, as may be seen in Fig. 167. This section requires careful designing to insure its stability. The flat slab top must be designed as a beam to carry the earth load and the side walls must be strong enough to resist the lateral earth pressure. If the top is built in the form of a flat arch, the side walls must be strengthened to carry the thrust of the arch.

In some cases the flat top has been constructed of I-beams encased in concrete, but this method is not economical of steel as the I-beams are designed to carry the load while the concrete merely acts as a filler between the beams and as a protection to the steel. This method, although not economical of steel, has the advantage of making it possible to complete the sewer and backfill the trench more quickly than where the roof is a slab reinforced with bars. The steel beams can be placed very easily and quickly and do not require such constant inspection, as is the case with slabs reinforced with bars. It is claimed that in some cases this ease of construction will offset the additional cost of the steel, and in many cases where a large sewer is

built in a congested district, it is of considerable advantage to be able to backfill the trench with the least possible delay.

The V-shaped invert is frequently used with the rectangular section on account of its suitability for low flows.

Semi-circular Section.—This type of sewer, examples of which are shown in Fig. 164, has been employed rather extensively in New York city and vicinity. Its most frequent use has been for outfall sewers crossing low land, where the natural surface of the ground is largely below the top of the sewer arch and in places even below the invert. Like the other arched sections, the invert must be firm and well designed to support the thrust of the arch. Two of these sections are often built side by side as twin sewers, instead of one large sewer, in order to save head room.

As a rule, the section requires a larger amount of masonry for its capacity than some of the other types. The hydraulic properties are not so advantageous as those of the rectangular section, which, in the vicinity of New York City, has been used largely to replace the semi-circular section. The semi-circular section requires a wider trench and more extensive foundations for equal capacity and height than most of the other types.

Sections with Cunette.—Various types of sewers have been constructed with a special dry-weather channel or cunette in the invert. This type, although used extensively in France and Germany, has been employed but little in the United States. The most notable example is in the trunk sewers at Washington, D. C., Fig. 168c.

This section requires additional masonry in the invert and a greater depth of trench, but has the advantage of providing a good channel in which self-cleansing velocities may be maintained when the flow is small.

Double and Triple Sections.—Where outfall sewers are located in thickly settled districts, and the available head room is seriously limited, it sometimes is of great advantage to divide the section up into two or more waterways joined together side by side in one structure. In other cases, where a storm-water sewer is constructed above or below a large sanitary sewer, it may be more economical to build both waterways in one structure, one over the other. Representative types of such structures are shown in Figs. 170, 171 and 172.

SELECTION OF TYPE OF SEWER

The selection of the type of sewer depends upon a number of conditions all of which must be carefully considered and balanced in the choice of the best type to build. In general, that sewer is the best which, for the least cost per linear foot, will be easy to maintain in operation and

will have the requisite stability to withstand the external and internal forces. In the following paragraphs a number of the principal items to be considered are enumerated in detail.

Hydraulic Elements.—The theoretically best cross-section for a sewer, from the point of view of hydraulics, with a given grade, s, and carrying a uniform quantity of water per unit of time, is the semicircle for an open channel and circle for a closed channel, both running full; because the hydraulic mean radius, r, has its greatest value for these sections and consequently the empirical coefficient, c, in the Chezy formula $v = c\sqrt{(rs)}$ for velocity of flow, is greatest.

This theoretical advantage is somewhat reduced by the fact that the flow in sewers is not uniform but is constantly changing in depth,

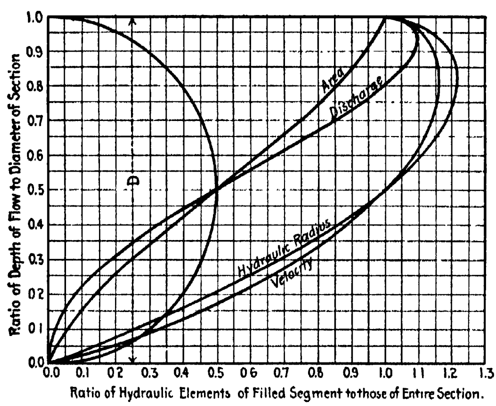


FIG. 133.—Hydraulic elements of circular section by Kutter's formula.
$n = 0.015$; $s = 0.005$; $D = 1$ ft. Area $= 0.785D^2$; Wetted Perimeter $= 3.1416D$; Hydraulic Radius $= 0.250D$.

and therefore the minimum velocity is an important feature. The circular section is not as advantageous as the egg shaped for low flows. In some cases a small semi-circular channel has been constructed in a V-shaped invert to carry the minimum flow. Assuming a certain minimum velocity is to be maintained with a given minimum quantity of sewage, the diameter of the small semi-circular channel required in the invert to carry this flow can be readily computed.

With very small quantities of sewage the diameter of the semi-circle becomes gradually smaller until finally the invert, instead of having two converging slopes with a depressed semi-circular channel at their junction, become actually a V-shaped section. In this case if h is the depth of the sewage and B is the angle between the sloping invert

bottom and the horizontal, the cross-section, A, is $h^2 \cot B$. The wetted perimeter, p, is $2h/\sin B$ and the hydraulic mean radius $r = \sqrt{(0.25\,A \times}$

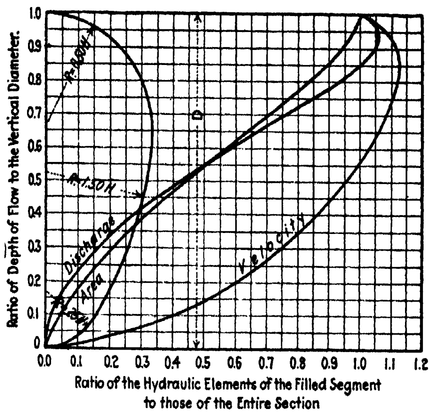


FIG. 134.—Hydraulic elements of egg-shaped section.

$n = 0.015$; $s = 1/1{,}600$; $H \doteq 4$ ft.; $D = 6$ ft. $= 1.254$ diam. equiv. circle; $H = 0.836$ diam. equiv. circle; $A = 1.1485\,H^2 = 0.5105D^2$; $R = 0.2897H = 0.1931D$.

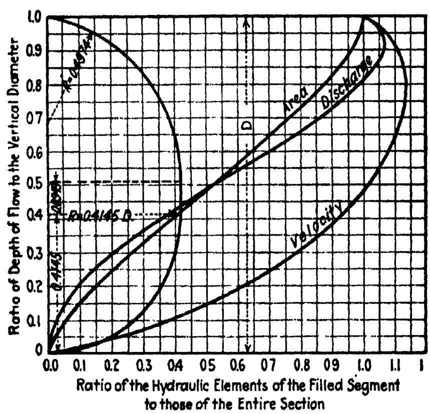


FIG. 135.—Hydraulic elements of gothic section.

$n = 0.015$; $s = 1/1{,}500$; $H = 3$ ft.; Horizontal diameter $= H = 0.9167$ diam. equiv. circle; Vertical diameter $= D = 1.1056$ diam. equiv. circle; Area $= 0.9534H^2 = 0.6554D^2$; $R = 0.2737H = 0.2269D$.

$\cos B \sin B)$. By this expression, the value of r is greatest when B is 45 deg., or when the two slopes of the invert form a right angle.

The V-shaped invert with the circular arc at the junction has been

used to advantage with the rectangular sewer section and also with some of the other types.

Where the normal flow is equal to one-third or more of the maximum

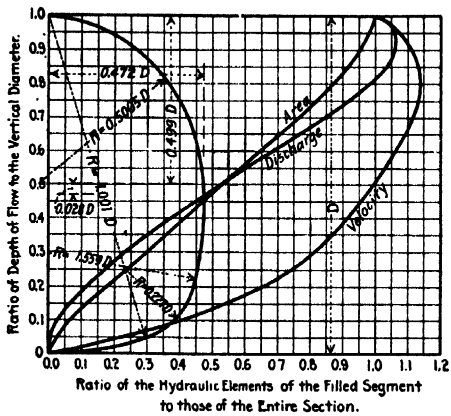


FIG. 136.—Hydraulic elements of basket-handle section.
n = 0.015; s = 1/3,000; D = 8 ft. 10 in.; Vertical diameter = D = 0.999 diam. equiv. circle; Area = $0.7862D^2$; $R = 0.2464D$.

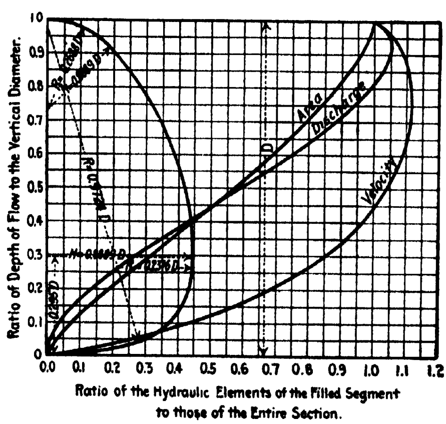


FIG. 137.—Hydraulic elements of catenary section.
n = 0.015; s = 1/3,000; D = 7.44; Vertical diameter = D = 1.063 diam. equiv. circle; Area = $0.70277D^2$; R = $0.23172D$.

flow, the circular type is the best from the point of view of velocity and carrying capacity, but there are other considerations which usually affect the form of the sewer and may dictate some other type.

In comparing one section with another, it is important to study the relation between the depth of flow and the corresponding velocity and

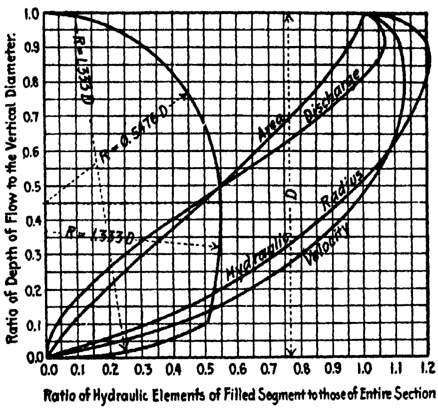


FIG. 138.—Hydraulic elements of horseshoe section, Wachusett type, by Kutter's formula.

n = 0.013; s = 0.0003; D = 7 ft.; Horizontal diameter, H = 7 ft. 8 in.; Area = 44.74 sq. ft. = $0.913D^2$; Wetted perimeter = 24.26 ft. = $3.466D$; Hydraulic radius = 1.841 = $0.263D$.

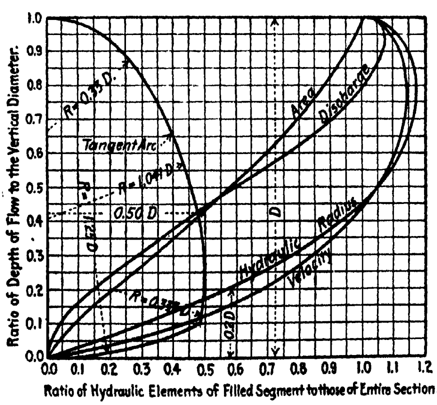


FIG. 139.—Hydraulic elements of Louisville semi-elliptic section by Kutter's formula.

n = 0.013; s = 0.0003; D = 7½ ft.; Area = $0.785D^2$; Wetted perimeter = $3.26D$; Hydraulic radius = $0.242D$.

discharge. The diagrams shown in Figs. 133 to 145 give, for each of the principal types of conduits, the ratio of each of the three hydraulic ele-

ments area, mean velocity and discharge, of the filled segment to that of the entire section, corresponding to any ratio of depth of flow to the

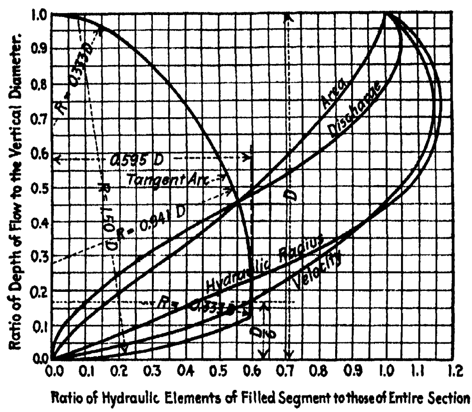


FIG. 140.—Hydraulic elements of special semi-elliptical section by Kutter's formula.

$n = 0.013$; $s = 0.0003$; $D = 7$ ft.; Horizontal diameter, H, = 8 ft. 4 in.; Area = $0.9D^2$ = 44.1 sq. ft.; Wetted perimeter = $3.508D$ = 24.56 ft.; Hydraulic radius = $0.256D$ = 1.79.

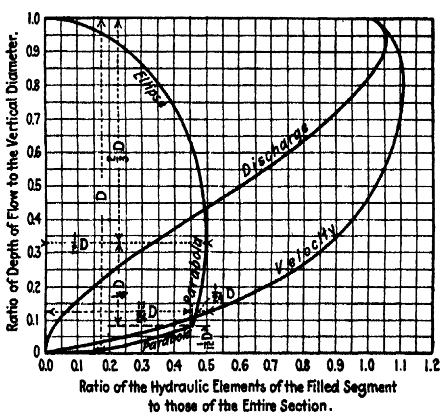


FIG. 141.—Hydraulic elements of Gregory's semi-elliptical section.

$n = 0.015$; $s = 0.0005$; $D = 10$ ft.; Area = $0.8176D^2$.

vertical diameter. Table 128 gives additional data regarding these same sections as well as others. Among other data, the table gives the wetted area and mean hydraulic radius of the filled sections in terms of the

TABLE 128.—DATA RELATING TO HYDRAULIC ELEMENTS OF SEWER SECTIONS

Type of sewer	Depth of flow	Area of wet section in terms of vertical diam. D	Mean hyd. radius in terms of vert. diam. D	Relative depth at point of maximum		Relative value at point of maximum		Basis of exact comparison			Vertical diam. in terms of "d" diam. of equiv. circle	Reference to figure number
				Velocity	Discharge	Velocity	Discharge	Vertical diam. D. in.	Slope S	Kutter's coef. n		
Circular	Full	$0.7854D^2$	$0.250D$	0.80	0.94	1.16	1.09	12.00	0.005	0.015	....	133
Circular	Full	$0.7854D^2$	$0.250D$	0.79	0.93	1.14	1.08	90.00	0.0003	0.013	....	....
Egg-shaped (standard)	Full[1]	$0.5105D^2$	$0.1931D$	0.85	0.94	1.13	1 06	72.00	0.000625	0.015	$1.254d$	134
Egg-shaped (standard)	⅔ Full[1]	$0.3359D^2$	$0.2105D$	....	....	....	....	....	....	....	....	....
Egg-shaped (standard)	⅓ Full[1]	$0.1262D^2$	$0.1377D$	....	....	....	....	....	....	....	....	....
Egg-shaped (new)	Full[2]	$0.4956D^2$	$0.1896D$	0.87	0.94	1.12	1.07	72.00	0 001–0.007	0.015	....	....
Egg-shaped (new)	⅔ Full[2]	$0.3210D^2$	$0.2049D$	....	....	....	....	....	....	....	....	....
Egg-shaped (new)	⅓ Full[2]	$0.1130D^2$	$0.1280D$	....	....	....	....	....	....	....	....	....
Gothic	Full[3]	$0.6554D^2$	$0.2269D$	0.79	0.92	1.13	1.07	43.42	0.00067	0.015	$1.1056d$	135
Basket handle	Full[3]	$0.7862D^2$	$0.2464D$	0.82	0.92	1.14	1.07	106.00	0.00033	0.015	$0.999d$	136
Catenary	Full[3]	$0.7028D^2$	$0.2317D$	0.77	0.93	1.11	1.06	89.28	0.00033	0.015	$1.063d$	137
Parabolic	Full	$0.7440D^2$	$0.2245D$	0.78	0.94	1.10	1.04	88.00	0.001	0.013	$1.0481d$	142
Semi-elliptical (Gregory's)	Full[4]	$0.8176D^2$	$0.2487D$	0.80	0.93	1.11	1.06	120.00	0.0005	0.015	....	141
Semi-elliptical (Louisville)	Full	$0.785D^2$	$0.242D$	0.77	0.93	1.14	1.07	90.00	0.0003	0.013	$1.000d$	139
Semi-elliptical (special)	Full	$0.900D^2$	$0.256D$	0.74	0.92	1.15	1.05	84.00	0.0003	0.013	$0.934d$	140
Five-centered (St. Louis)	Full[5]	$0.9747D^2 + 0.5683D - 0.1343$	....	....	....	....	....	....	....	0.013	....	....
Horse-shoe (Wachusett)	Full[3]	$0.913D^2$	$0.263D$	0.82	0.92	1.13	1.06	126.00	0.0004	0.015	$0.939d$	138
Horse-shoe (Croton)	Full[3]	$0.850D^2$	$0.256D$	0.80	0.93	1 12	1.07	162.36	0.0004	0.015	$0.965d$	....
Horse-shoe (Boston)	Full	$0.8293D^2$	$0.2539D$	....	....	....	....	102.00	0 0004	0.015	$0.954d$	....
Horse-shoe (St. Louis)	Full[5]	$1.0139D^2$	$0.2750D$	....	....	....	....	....	....	0.013	....	....
Semi-circular	Full	$1.2697D^2$	$0.2946D$	0.80	0.92	1 14	1.07	110⅛	0.001	0.013	$0.8009d$	145
U-shaped	Full	$0.6438D^2$	$0.2047D$	Almost full depth		1.23	1.22	30.00	0.002	0.013	$1.1353d$	143
Rectangular	Full	$1.3125D^2$	$0.2865D$	Almost full depth		1.30	1.30	.....	0.001	0.013	$0.7968d$	144
Rectangular	$1\frac{1}{2}D$	$1.1875D^2$	$0.4074D$	....	....	....	....	....	0 001	0.013	$0.7594d$	....

[1] From "Hydraulic Tables" by P. J. Flynn. [2] From "Hydraulic Diagrams and Tables" by Garrett. [3] From "Hydraulic Diagrams" by Swan and Horton. [4] From Gregory. *Eng. News*, March 12, 1914. [5] From W. W. Horner, St. Louis, Mo.

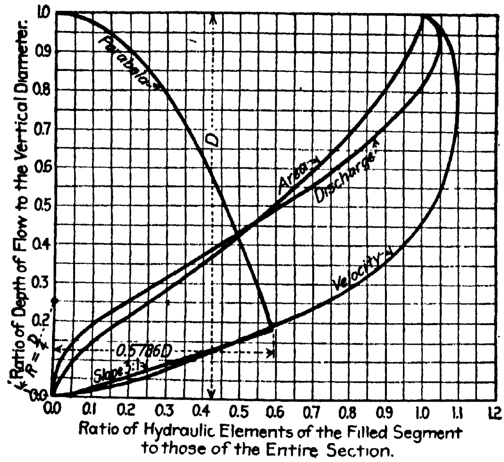


Fig. 142.—Hydraulic elements of parabolic section.
$n = 0.013$; $s = 0.001$; $D = 7$ ft. 4 in.; Area $= 0.744D^2$: Hydraulic radius $= 0.2245D$.

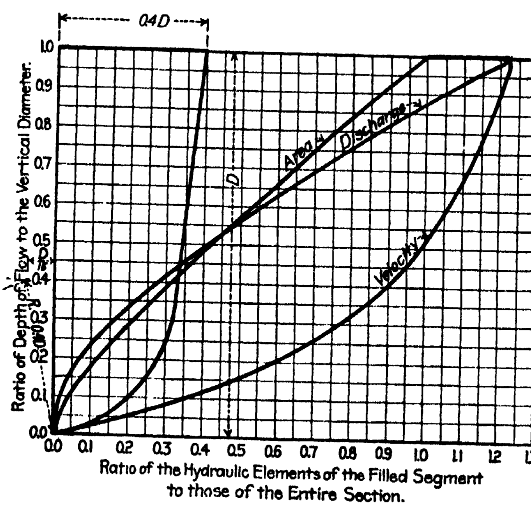


Fig. 143.—Hydraulic elements of U-shaped sections.
$n = 0.013$; $s = 0.002$; $D = 2$ ft. 6 in.; Area $= 0.6438D^2$; Hydraulic radius $= 0.2047D$.

vertical diameter, and the relative value of the vertical diameter of the section in terms of the diameter of the equivalent circular section. By

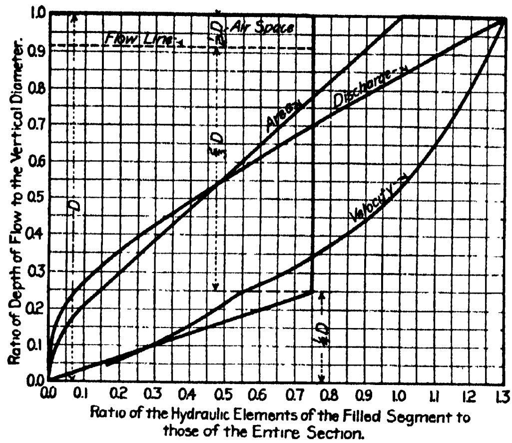


FIG. 144.—Hydraulic elements of rectangular section.

$n = 0.013$; $s = 0.001$; $D = 6$ ft.

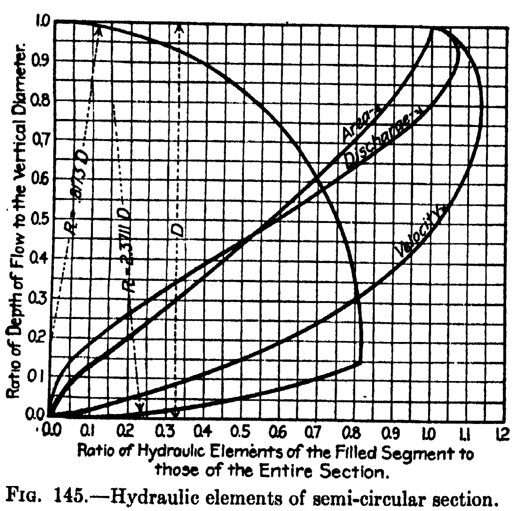


FIG. 145.—Hydraulic elements of semi-circular section.

$n = 0.013$; $s = 0.001$; $D = 9$ ft. $2\frac{1}{8}$ in.; Area $= 1.2697D^2$; Hydraulic radius $= 0.2946''$

equivalent section is meant that section which has the same carrying capacity for a given size, slope and friction factor, but not the same area.

The table, also gives the actual size, slope and friction factor upon which the table and curves were computed, which, although strictly correct only for the data given, are sufficiently close for other sizes, slopes and friction factors to be of general use, and as a rule the difference may be neglected.

That there can be a slight difference in the hydraulic elements for various depths of flow, depending on the size of the section for which the diagram is computed, is shown by the first two lines of Table 128. The first line gives some of the hydraulic elements of a circular section based on a 12-in. pipe where s = 0.005 and n = 0.015. The second line was computed on the basis of a circular section 7 ft. 6 in. in diameter; s = 0.0003, and n = 0.013. The change in the value of n was made on account of the authors' practice which assumes 0.015 for pipe sewers and 0.013 for concrete sewers. The slopes were also changed in order to approximate the slopes usually adopted in practice for the respective sizes.

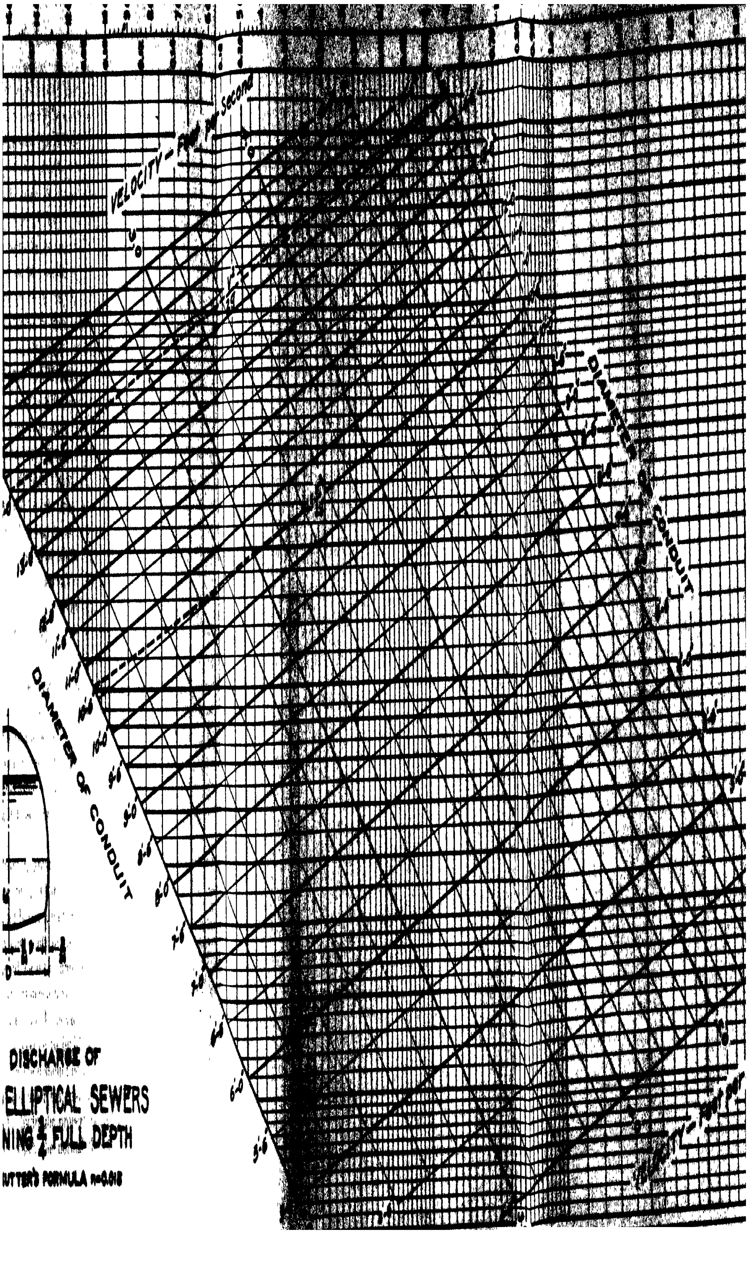
As previously stated, that section is the best which for varying depths of flow maintains the hydraulic mean radius most nearly constant.

Where high velocities occur another element is introduced in the way of erosion of the invert and sides of the sewer, which may require special construction to prevent serious wear and ultimate destruction.

Construction and Available Space.—The method of construction of a sewer, whether in open cut or in tunnel may have an important influence on the selection of the type. In tunnel work, especially, it is desirable to have a section which will utilize to the best advantage all of the space inside the tunnel bracing. In earth tunnels, where the common form of timbering is used, the catenary or semi-elliptical sections conform readily to the available space. In rock tunnels, the circular or horse-shoe sections are apt to be more advantageous. If the sewer is built in open cut, the section will be influenced by its ability to carry the earth loads.

Where the excavation is in rock or firm soil, it is possible to shape the bottom of the trench to conform to the shape of the invert of the sewer and thereby save considerable thickness of masonry for such types as the circular or egg-shaped sections. If the excavation is in soft material, where the bottom of the trench must necessarily be flat, or if the sewer is to be built on piles or a timber platform, considerable additional masonry will be required for the circular or egg-shaped sewers, but this can be avoided by using one of the other types.

The amount of space available for a sewer may be exceedingly limited. Sometimes the head room is limited because of the proximity of the grade of the sewer to the surface of the street, sometimes the side room is limited because of adjacent structures, and then again the available depth may be limited on account of tide water or other circumstances


VELOCITY—Feet per Second
DIAMETER OF CONDUIT
DIAMETER OF CONDUIT
DISCHARGE OF
ELLIPTICAL SEWERS
NING $\frac{3}{4}$ FULL DEPTH
UTTER'S FORMULA n=0.015

which control the allowable depth of the hydraulic grade line. The rectangular section has proved one of the most useful for such conditions, although the horse-shoe section, with the horizontal or vertical diameters adjusted to meet the conditions, has been largely used. In a few cases the full elliptical section has also been used in restricted places. Where the hydraulic grade line depth is limited, it is desirable to use a sewer section which will carry the maximum and minimum flows with the least variation in depth of flow. The catenary, parabolic, semi-elliptical and rectangular sections are especially suitable for this purpose, as the center of gravity of the wetted area is comparatively low down from the crown in contrast to the circular section. The semi-circular section has also proved useful in this connection, although the rectangular section is being used instead in the more recent work of this character.

Cost of Excavation and Materials.—The cost of excavation required by one type as compared with another should be carefully considered, for if the excavation is in earth in a deep trench, it will probably be cheaper to use a narrower deeper section and thereby save considerable width of excavation, even though the depth of excavation be slightly increased. This will be especially true in a deep rock trench where it may be found of advantage to use a narrow rectangular section having a height 1-1/2 to 2 times the width. For a sewer built in very shallow cut, or practically on the surface of the ground, a wider section will be advantageous, because little additional cost is incurred by increasing width whereas greater depth may increase materially the cost of excavation. Furthermore the cost of an embankment over a wide section will generally be less, because of reduced height and narrower side slopes. The parabolic or delta section is especially useful for crossing low lands where the sewer is largely out of the ground and must be covered by an embankment. The semi-circular section has also been much used for this same purpose, but has been superseded more recently by the rectangular section, having a width about 1-1/2 times its height.

In former years a great many sewers were constructed of quarry stone or large cobbles, but in recent years other materials have proved less expensive and better adapted to this type of construction and very few sewers are now built in this manner. The cost of brick varies greatly in different localities and this may influence to a large extent the type of construction selected. In general, concrete is more desirable than brick, but where brick masonry can be had much cheaper than concrete it may be advisable to build the sewer of brick. The object in designing a sewer section should be to obtain one in which the quantity of the masonry and other materials is a minimum consistent with the requisite stability, hydraulic properties and other considerations.

For sewers in which the normal flow is at least one-third of the

maximum flow, it has been found that the semi-elliptical section is very economical in masonry, and at the same time provides for the other requirements.

Stability.—Where a sewer is constructed in open trench, the structure must be designed to carry the earth or trench load as well as any superimposed load. The circular arch is not as strong as either the Gothic, the parabolic, or the semi-elliptical arch. The semi-circular arch depends to a great extent upon the lateral pressure of the sides of the trench and also to a certain extent on the lateral resistance or passive pressure of the earth backfilling, although this can be obviated by increasing the thickness of the side walls or abutments. The semi-circular sections obviate part of this difficulty by omitting the side walls and resting the springing line of the arch directly on the invert or foundation. In a rock trench the resistance of the sides of the trench is so great that the side walls of the sewer can be greatly reduced in thickness, the thrust of the arch being carried directly into the walls of the trench. In this latter case a very flat arch can be used to advantage.

Imperviousness.—Where a sewer is to be constructed under a river bed or below the water table, it may be of particular importance for the walls of the sewer to be impervious. To this end, if the sewer is built of concrete, it is desirable to insert longitudinal reinforcing bars in the concrete, with a total area of 0.2 to 0.4 per cent. of the sectional area of the concrete in order to distribute the stress throughout the length of the sewer barrel and thereby prevent the formation of large cracks which would permit leakage. Unless the cracks are very small there may be some danger of corrosion due to the water passing through them and coming in contact with the reinforcement. This might in time weaken the structure.

While the possibility of leakage or infiltration does not ordinarily determine the shape of the waterway of a sewer, it is worthy of consideration when the selection is to be made. For example, if a sewer is to be built below the water table it may be well to adopt a section which is least likely to crack, whereas under other conditions the advantages of a different section might be sufficiently great to warrant its use even though small arch cracks are to be expected. The stability of the horseshoe section depends to a certain extent on the lateral pressure of the earth backfilling, and on that account, the semi-circular arch is apt to crack and may produce unsatisfactory conditions, not only because of leakage into the sewer, but especially on account of the rusting of the steel reinforcement.

SELECTION OF SIZE OF SEWER

In Chapters V and VIII methods are given by which the quantity of sewage and storm water for which the sewers are to be designed, can be

estimated. In determining the size of sewer to carry this estimated quantity, an additional factor of safety is often allowed by computing the sewer as flowing less than completely full, as one-half or two-thirds full. Such an allowance does not seem to be logical in most cases, for uncertainties as to the quantity of sewage produced and the hourly, daily and seasonal variations should be provided for in estimating these quantities, the sewer being designed to carry them without further allowances, its capacity corresponding to the maximum estimated quantity of sewage.

Designs of rectangular and U-shaped sewers, and to a less extent other types, should always be based on the maximum capacity of the sewer and not upon its capacity when completely full. As can be seen from Figs. 143 and 144, both the velocity and discharge are materially reduced when the inside perimeter of the sewer becomes completely wet, owing to the reduction in the hydraulic mean radius. Sewers of these types should always be designed with an air space at the top, that they may develop their maximum capacity.

Hydraulic Diagrams and Tables.—Diagrams giving the discharge of circular conduits can be used to compute the velocity and discharge in the case of conduits of other shapes, provided the hydraulic mean radius of the section in question is known. Any two sewers having the same hydraulic mean radius and constructed on the same slope, will have the same velocity, but not necessarily the same discharge, owing to the difference in the area of the sections.

If we know the hydraulic mean radius of a special section, as, for example, a parabolic section, we can find the corresponding velocity from the diagram for circular conduits for any specified slope; and from the product of the velocity thus obtained times the area of the parabolic section the corresponding discharge of that section can be computed.

Where considerable work is to be done with one type of sewer of different sizes, it will be found a great convenience to construct a diagram for it, in order to save computations. A diagram of this kind is shown in Fig. 146, giving the discharge of semi-elliptical sewers, Gregory's type, running 3/4 full depth, by Kutter's formula for $n = 0.015$. This diagram was furnished by John H. Gregory and was published in *Engineering News*, March 12, 1914, from which the following paragraphs are quoted:

"The velocity in and corresponding discharge of semi-elliptical sewers of the section shown, when running three-quarters full depth, can be readily obtained from the diagram. The diagram is based on Kutter's formula, with $n = 0.015$, and covers the range in diameters and velocities ordinarily met with in practice. The diagram is practically self-explanatory but it may be said that from any point inside the diagonal lines the corresponding diameter, velocity, slope and discharge can be read.

"It is often desirable to know the velocity head and the loss of head at entrance, or the sum of the two, and either or all of these quantities can be obtained from the diagram. Thus, to find the head required to produce a velocity of 3 ft. per second it is only necessary to find the intersection of the velocity line 3 with the dotted line marked $V^2/2g$ and read the velocity head corresponding thereto on the scale marked 'Slope in Feet per 1000,' or 0.14 ft. The loss of head at entrance would be found by dividing the velocity head by 2, assuming that the loss of head at entrance would be $0.5v^2/2g$. The sum of the velocity head plus the loss of head at entrance is found in the same manner as the velocity head alone, except that the dotted line marked $1.5\ v^2/2g$ is to be used in finding the intersection with the velocity line. For a velocity of 3 ft. per second the value of $1.5v^2/2g$ is seen to be 0.21 ft."

Data of this character can also be arranged in the form of a table, similar to Table 129, which gives the values of the hydraulic elements of the Boston type of horse-shoe section, as computed by F. A. Lovejoy of the Boston Sewer Department. These values are based on Kutter's formula for $n = 0.013$. The Boston type of horse-shoe section is shown in Fig. 132c. The values in the table multiplied by $\sqrt{s}$, (s = the slope), will give the corresponding discharge of the sewer flowing full. The form of this table is that given by P. J. Flynn, "Hydraulic Tables," Van Nostrand Science Series.

Equivalent Sections.—A diagram designed by Frank Allen and Otis F. Clapp, for use in the City Engineer's office at Providence, R. I., was published in *Eng. Record*, Oct. 8, 1904. This diagram, Fig. 147 shows the dimensions of equivalent horse-shoe and circular conduits flowing full, based on Kutter's formula for $n = 0.013$. The form of the horse-shoe section is shown in the figure, H being the vertical diameter, W the horizontal diameter; the radius of the side walls, $2W$, and the radius of the invert $2W$. By equivalent conduits is meant conduits having equal carrying capacities but not necessarily equal areas. In this type of horse-shoe section, the arch is always semi-circular. The limiting cases covered by this diagram are a section having only arch and invert, in which H is 0.5635 W and a section in which $H = W$.

The following modified form of Kutter's formula given in Swan & Horton's "Hyradulic Diagrams" was used in computing this diagram.

$$V = \left(\frac{Z}{1 + \frac{x}{\sqrt{R}}}\right)\sqrt{RS}$$

in which V is the mean velocity of flow, R the hydraulic mean radius, S the slope, and x and Z are coefficients. The quantities x and Z vary but slightly between wide limits in the value of S, and may therefore be considered approximately constant within such limits. With n = 0.013 for S between the limits of 0.001 and 0.010, $x = 0.551$ and Z

TABLE 129.—VALUES OF HYDRAULIC ELEMENTS OF HORSE-SHOE SEWER (BOSTON TYPE, FIG. 132c). COMPUTED BY BOSTON SEWER DEPT. BY KUTTER'S FORMULA, $n = 0.013$

Diameter in ft.	in.	Area = A in sq. ft.	Hydraulic mean radius R in ft.	Wetted perimeter in ft.	For discharge, $Ac\sqrt{R}$ cu. ft. per sec.
3	0	7.463706	0.7615	9.800487	719.3024
3	1	7.86707	0.7818	10.06183	773.2628
3	2	8.33355	0.8046	10.35684	835.7266
3	3	8.75948	0.8250	10.617195	893.5074
3	4	9.19592	0.8453	10.87854	953.5299
3	5	9.66982	0.8654	11.172556	1019.5322
3	6	10.15892	0.8884	11.433902	1092.2659
3	7	10.63162	0.9090	11.69514	1158.9166
3	8	11.17067	0.9325	11.989229	1239.8739
3	9	11.65995	0.9517	12.250609	1311.9132
3	10	12.16593	0.9731	12.491844	1380.7469
3	11	12.74634	0.9952	12.806934	1479.1253
4	0	13.2668	1.007	13.0672	1559.3082
4	1	13.8078	1.0434	13.32854	1653.6676
4	2	14.4215	1.0586	13.6225	1746.5319
4	3	14.9771	1.0787	13.8849	1834.4428
4	4	15.54937	1.0992	14.1452	1930.9992
4	5	16.20452	1.1222	14.4392	2040.3911
4	6	16.79332	1.1424	14.7006	2131.3518
4	7	17.39871	1.1628	14.9619	2243.1929
4	8	18.08703	1.1855	15.25595	2361.4387
4	9	18.62607	1.2003	15.517300	2453.575
4	10	19.34756	1.2262	15.77764	2585.1106
4	11	20.07735	1.2491	16.07265	2715.7896
5	0	20.7325	1.2693	16.33400	2834.0581
5	1	21.4042	1.2897	16.5953	2959.2771
5	2	22.16718	1.3125	16.8893	3101.5844
5	3	22.8550	1.3326	17.1507	3228.2596
5	4	23.5604	1.3531	17.4120	3362.0143
5	5	24.3648	1.3760	17.706	3518.0714
5	6	25.0863	1.3962	17.9674	3655.9182
5	7	25.8244	1.4166	18.2287	3801.3702
5	8	26.6619	1.4393	18.5227	3963.9995
5	9	27.4176	1.4596	18.7841	4116.7821
5	10	28.1879	1.480	19.0454	4270.7136

TABLE 129.—*Continued.*

Diameter in ft.	in.	Area = A in sq ft	Hydraulic mean radius R in ft.	Wetted Perimeter in ft.	For discharge, $Ac\sqrt{R}$ cu ft. per sec.
5	11	29.0679	1.5030	19.3394	4447.3226
6	0	29.8548	1.5231	19.6008	4612.3474
6	1	30.655	1.5434	19.8621	4774.4581
6	2	31.5631	1.5664	20.1561	4951.4730
6	3	32.3924	1.5865	20.4175	5138.4725
6	4	33.2310	1.6070	20.6788	5313.4840
6	5	34.1837	1.6299	20.9728	5517.6147
6	6	35.0379	1.6500	21.2342	5699.9172
6	7	35.908	1.6705	21.495	5879.082
6	8	36.8955	1.6932	21.7895	6105.650
6	9	37.7829	1.7133	22.0509	6301.0709
6	10	38.6868	1.7340	22.31019	6501.3060
6	11	39.7151	1.7568	22.6062	6730.3951
7	0	40.6357	1.7770	22.8676	6944.2581
7	1	41.5728	1.7975	23.1289	7158.2220
7	2	42.6343	1.8201	23 4229	7402.0088
7	3	43.5880	1.8404	23.6843	7621.617
7	4	44.4738	1.8573	23 9456	7846.149
7	5	45.6562	1.8836	24.2396	8099.739
7	6	46.6481	1.9039	24.5010	8330.442
7	7	47.6482	1.9257	24.7623	8576.752
7	8	48 7794	1.9472	25.0563	8837.631
7	9	49.8098	1.9674	25.3177	9078.188
7	10	50.8443	1.9876	25.5888	9334.582
7	11	52.0186	2.0107	25.8730	9619.087
8	0	53.0752	2.0311	26.1344	9892.649
8	3	56.4438	2.0943	26.9511	10723.857
8	6	59.9169	2.1576	27.7678	11601.486
8	9	63 4912	2.2214	28.5845	12534.523
9	0	67.1733	2.2848	29.4012	13534.528
9	3	70.9549	2.3487	30.2179	14499.966
9	6	74.8443	2.4119	31.0346	15552.709
9	9	78.8332	2.4751	31.8513	16665.544
10	0	82.9300	2.5361	32.668	17796.910
10	3	87.1262	2.6021	33 4847	19027.025

TABLE 129.—*Continued.*

Diameter in ft.	in.	Area = A in sq. ft.	Hydraulic mean radius R in ft.	Wetted Perimeter in ft.	For discharge $Ac\sqrt{R}$ cu. ft. per sec.
10	6	91.4303	2.6655	34.3014	20248.307
10	9	95.8339	2.7294	35.1181	21561.979
11	0	100.3453	2.7927	35.9348	22903.471
11	3	104.9562	2.8559	36.7515	24266.969
11	6	109.6749	2.9199	37.5682	25718.536
11	9	114.4931	2.9831	38.3849	27226.626
12	0	119.4192	3.0464	39.2016	28757.289
12	3	124.4468	3.1103	40.0183	30386.664
12	6	129.5781	3.1736	40.8350	32032 044
12	9	134.8110	3.2367	41.6517	33716 985
13	0	140.1517	3 3007	42 4684	35479 457

= 181.69, with sufficiently close approximation; while for S between 0.010 and 1.00, $x = 0.542$ and $Z = 181.02$.

In order to describe the method of using the diagram in Fig. 147 the following example is quoted from *Eng. Record*, Oct. 8, 1904:

"Required a horse-shoe shape 78 in. high, equivalent in discharging capacity when flowing full to a 96-in. circular section. Find 78 at the left and 96 at the bottom of the diagram; trace the horizontal line through 78 to its intersection with the vertical through 96, which falls upon a height diagonal numbered 65; then trace along the 78 horizontal again, to the right or left, as the case may require, until the 65 width diagonal is met; then look to the top and find 120 for the width of the horse-shoe. All dimensions are given in inches. A 78 × 120-in. section of the height shown is equivalent in flowing capacity to a 96-in. circle."

For sections larger than those plotted on the diagram a convenient fraction, such as one-third, of the dimensions may be taken, and the results increased three times to obtain the desired figures.

SELECTION OF CROSS-SECTIONS

In selecting the dimensions of the masonry section to provide sufficient thickness to prevent excessive stresses in the masonry and at the same time be economical of material, it is unwise to reduce the thickness to theoretical limits on account of the uncertainty as to the quality of work obtainable. The relative saving by using extremely thin sections with high stresses is small and is usually false economy. For masonry sewers 5 ft. in diameter and less the thickness of the best section will often depend more on the minimum thickness allowable on account of construction methods than on the stresses developed in the section. For plain and reinforced concrete sewers, a minimum crown thickness of 5 in.

is considered good practice but a thickness less than that amount is more or less questionable, when the intention is to obtain first-class work.

Empirical Formulas for Thickness of Arches.—In selecting the dimensions of a trial arch section, some of the following formulas may be of assistance. They should not be relied upon, however, to give the final section. The formulas are only approximate and do not take into account many of the conditions which should govern the design of an arch. The majority were developed for use in designing arches having spans of 20 ft. or more and on that account may be less accurate for arches of smaller span.

In the following notation all dimensions are in feet.

t_c = thickness of arch at crown.
t_s = thickness of arch at springing line.
S = clear span of arch.
R = rise of intrados
r = radius of intrados at particular point under consideration.
F = height of earth fill over crown of extrados.

F. F. Weld in *Eng. Record*, Nov. 4, 1905, gives the following:

"The writer has devised the following equation, based upon a study of all available data upon the subject and his own experience in designing arches for a great variety of conditions. He believes it a safe guide for all ordinary conditions of span and load:

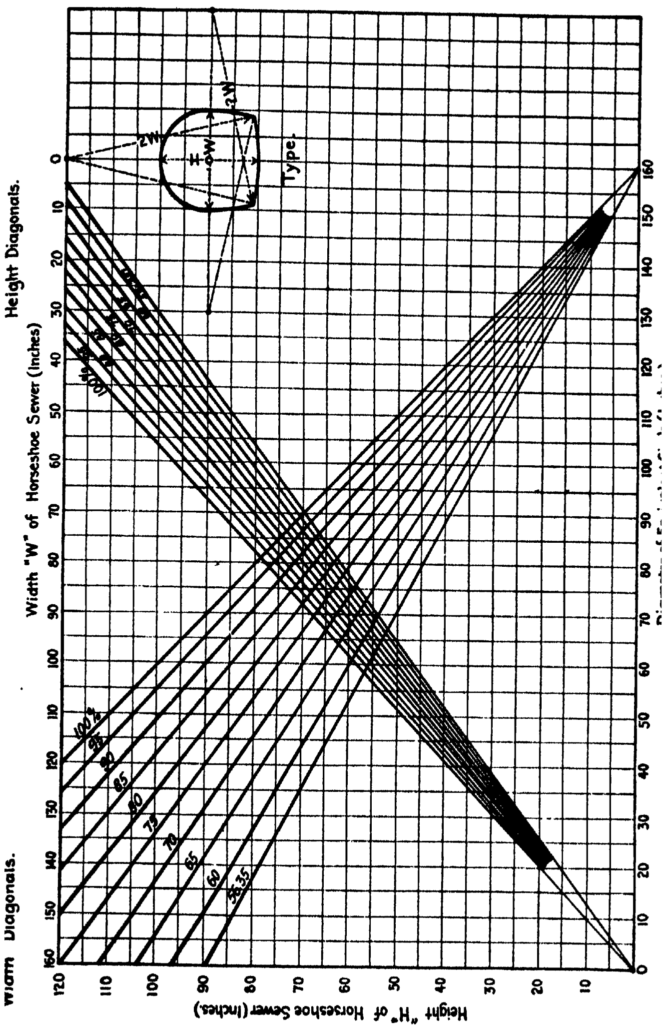
$$t_c = \tfrac{1}{12}(\sqrt{S} + 0.1S + 0.005L_l + 0.0025D)$$

where L_l = live load uniformly distributed, and D = weight of earth fill over the crown, both in pounds per square foot. "The arch ring at the quarter points should have a depth of from $1\frac{1}{4}$ t_c to $1\frac{1}{2}$ t_c, depending upon the curve of the intrados."

W. B. Fuller has developed the following rule for unreinforced concrete used where sheeting is not required: Make crown thickness a minimum of 4 in., and then 1 in. thicker than diameter of sewer in feet. Make thickness of invert same as crown plus 1 in., but never less than 5 in. Make thickness at springing line $2\frac{1}{2}$ times thickness of crown, but never less than 6 in. If ground is soft or trench is unusually deep, these thicknesses must be increased according to experienced judgment. (Taylor and Thompson, "Concrete, Plain and Reinforced," 1909, p. 684).

Taylor and Thompson, in "Concrete, Plain and Reinforced," p. 541, state that the Weld formula gives fairly correct results in ordinary cases.

"Obviously the thickness for a hingeless arch should increase from the crown to the springing. The radial thickness of the ring at any section is frequently made equal to the thickness at the crown multiplied by the secant of the angle which the radial section makes with the vertical. For a three-centered intrados and an extrados formed by the arc of a circle, these


Height Diagonals.
Width "W" of Horseshoe Sewer (Inches)
Width Diagonals.
Height "H" of Horseshoe Sewer (Inches)
Type.
2W
H
W

trial curves may be at the quarter points a distance apart of $1\frac{1}{4}$ to $1\frac{1}{2}$ times the crown thickness and at the springings 2 to 3 times the crown thickness."

Baker's "Masonry," tenth edition, p. 643, gives a number of different empirical formulas for determining the sections of a masonry arch, from which the following are quoted:

"*Trautwine's Formula* for the depth of the keystone for a first-class cut-stone arch, whether circular or elliptical, is

$$t_c = \tfrac{1}{4}\sqrt{(r + \tfrac{1}{2}S)} + 0.2$$

For second-class work, this depth may be increased about one-eighth part; and for brick work or fair rubble, about one-third.

"*Rankine's Formula* for the depth of keystone for a single arch is

$$t_c = \sqrt{0.12r}$$

and for tunnel arches, where the ground is of the firmest and safest,

$$t_c = \sqrt{(0.12R^2/S)}$$

and for soft and slipping materials twice the above. The segmental arches of the Rennies and the Stephensons, which are generally regarded as models, have a thickness at the crown of from 1/30 to 1/33 of the span, or of from 1/26 to 1/30 of the radius of the intrados.

"*Dejardin's Formulas,* which are frequently employed by French engineers, are as follows:

For circular arches,

$$\text{if } R/S = 1/2, \qquad t_c = 1 + 0.100r$$
$$\text{if } R/S = 1/6, \qquad t_c = 1 + 0.050r$$

For elliptical and basket-handle arches,

$$\text{if } R/S = 1/3, \qquad t_c = 1 + 0.070r$$

By Dejardin's formulas the thickness at the crown decreases as the rise increases—as it should.

"*Croizette-Desnoyers,* a French authority, recommends the following formulas:

$$\text{if } R/S > 1/6, \qquad t_c = 0.50 + 0.28\sqrt{2r}$$
$$\text{if } R/S = 1/6, \qquad t_c = 0.50 + 0.26\sqrt{2r}$$

American Civil Engineers' Pocketbook, first edition, p. 623–4, gives the following formulas for the approximate thickness of a masonry arch at the crown for spans under 20 ft.:

First class ashlar	$t_c = 0.04(6 + S)$
Second class ashlar or brick	$t_c = 0.06(6 + S)$
Plain concrete	$t_c = 0.04(6 + S)$
Reinforced concrete	$t_c = 0.03(6 + S)$

The thickness of masonry at the springing line may be computed in the following manner from the crown thickness, as given by the above formulas.

"Add 50 per cent. for circular, parabolic and catenarian arches having a ratio of rise to span less than 1/4. Add 100 per cent. for circular, parabolic, catenarian and three-centered arches having a ratio of rise to span greater

than 1/4. Add 150 per cent. for elliptical, five-centered and seven-centered arches. These thicknesses should be measured along radial joints."

It is also stated that the crown thicknesses, computed by the above formulas, should be increased about 60 per cent. for culverts under a high fill and about 25 per cent. for railroad arches.

Frye, in his "Civil Engineers' Pocketbook," 1913, p. 766, states that the following formulas give very close results for first-class concrete and cut-stone work:

For highway bridges, $t_c = \sqrt{0.01S\left(\frac{S}{R} + 3\right)} + 0.15$

For high highway embankments or for railroad bridges, $t_c = \sqrt{0.01S\left(\frac{S}{R} + 4\right)} + 0.20$

For high railroad embankments, $t_c = \sqrt{0.01S\left(\frac{S}{R} + 5\right)} + 0.25$

For all cases $t_s = t_c[1 + 0.002(S + 2R)]$

Baldwin Latham, in his "Sanitary Engineering," second edition, offers the following formula as being convenient for determining the proper thickness of the brickwork of sewers: "Thickness of brickwork in feet = $0.01dr$, where d = depth of excavation and r = external radius of sewer."

Watson, in his "Sewerage Systems," p. 86, gives Depuis' formula as being in use in France for computing the thickness of brickwork for sewers:

For sewers under sidewalks, this is

$$t_c = t_s = 0.2\sqrt{S}$$

For sewers under carriage ways, this is

$$t_c = t_s = 0.2\sqrt{S} + 0.02F$$

Reuterdahl, in his "Reinforced Concrete Arches," 1908, p. 43, prescribes two formulas for proportioning arches. The first, the Weld formula, has already been given, and the second, the D. B. Luten formula, is as follows:

$$t_c = \frac{3S^2(R + 3F)}{4000R - S^2} + \frac{L_lS^2}{30{,}000R} + \frac{L_m(S + 5R)}{150R} + 4$$

Where F = the depth of fill over the crown of the extrados, in feet.
L_l = Live load uniform in pound per square foot.
L_m = Moving load that will be concentrated on single track or single roadway over entire span in tons of 2000 lb.

Buel and Hill, in their "Reinforced Concrete," 1904, p. 104, suggest the following: "An approximate depth of the ring at the crown for reinforced-concrete arches may be found by the formula,

$$t_c = 0.0075(S + 10R)"$$

Howe, in his "Symmetrical Masonry Arches," first edition, p. 44, gives among other formulas, the majority of which have already been quoted, the following, designated as Perronet's formula for circular or elliptical arches (taken from paper by E. Sherman Gould, Van Nostrand's Mag., vol. xxix, p. 450.)

$$t_c = 1 + 0.035S$$

Parmley.—The following empirical formula was derived by Walter C. Parmley from a number of analyses of the stresses in sewer arches made in connection with the design of the Walworth sewer, Cleveland, Ohio (*Trans. Am. Soc. C. E.*, vol. lv, p. 357). Let t_s = the required thickness of the arch on a horizontal line through the center of the sewer, in feet, and S = the span or diameter of the sewer; then

$$t_s = \frac{S}{\frac{S}{14} + 2.572}$$

This formula is applicable to arches constructed of brick masonry.

Emile Low, in *Engineering News*, June 15, 1905, offers the following formula for the crown thickness of masonry arches:

$$t_c = \tfrac{1}{8}\sqrt{10(S - R) + 2h}$$

He states that the formula with a divisor of 6 instead of 8, as given, will closely approximate the crown depth of many modern structures.

One method of computing the thickness of the arch at the springing line has already been referred to in the paragraph quoted from Taylor and Thompson. This method assumes that the loads are vertical and that the horizontal component of the compression on the arch ring is constant.

Another formula for the thickness of the arch at the springing line or the thickness of the abutment at the springing line is that given by Trautwine, as follows:

$$t_s = 0.2r + 0.1R + 2.0$$

The formulas mentioned in the preceding paragraphs are based for the most part on existing structures, and on that account the use of these formulas may lead to safe results, for similar materials and conditions of load, although the factor of safety will be in doubt.

CHAPTER XII

EXAMPLES OF SEWER SECTIONS AND THE LOADS ON SEWERS

Sewer Sections Actually Used.—The designing engineer will derive much assistance from a study of sewer sections used by other engineers. The dimensions of many such sections are available, although so scattered through engineering literature as to make difficult a ready comparison of their salient features.

In cities where considerable sewer construction is in progress, it has often been found advantageous to formulate a set of standard sections for sewers of different sizes, thus making it unnecessary to prepare special designs for each sewer. These standard sections, especially the smaller sizes, have been based largely on the analysis of a number of sections previously adopted, and upon experience in their construction. They are valuable, therefore, as representing the judgment and experience of engineers with respect to sewers actually constructed, and as not necessarily being confined to theoretical lines.

The data relating to and the illustrations of sewer sections presented in the following pages, should be considered merely as furnishing to the designing engineer suggestions which he may find helpful in preparing designs for the particular work in hand. As the local conditions attending the construction of these sewers cannot be accurately known, it should not be assumed that any of them can be adopted without modification for the conditions surrounding the work in hand.

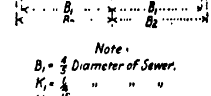


FIG. 148.—Louisville standard concrete section.

Standard Sewer Sections.—In Figs. 148 to 152, inclusive, and in Tables 130 to 134 are shown a number of sections adopted as standard in several cities.

Louisville, Ky.—The cross-sections of plain concrete sewers shown in Fig. 148, and Table 130, were prepared for the Commissioners of Sewerage of Louisville, Ky., J. B. F. Breed, Chief Eng. The dimensions given were based on what experience had shown to be a safe thickness of masonry under the conditions there existing. The minimum thickness at the crown and at the invert was fixed at 5 in. because of the practical

TABLE 130.—DIMENSIONS OF PLAIN CIRCULAR CONCRETE SEWERS, LOUISVILLE.

Dimensions of the sections										Quantity of concrete cu. yd. per lin. ft. sewer	
Diameter	H_1	H_2	B_1	B_2	K_1	K_2	R_1	R_2	R_3	Firm ground	Soft ground
24″	5″	5″	1′ 7½″	1′ 8¼″	6″	1′10½″	1′ 0″	1′ 6″	1′ 5″	0.13	0.15
27″	5″	5″	1′ 9⅝″	1′10¾″	6¾″	2′ 1 5/16″	1′ 1½″	1′ 8″	1′ 6½″	0.15	0.18
30″	5″	5″	2′ 0″	2′ 1½″	7½″	2′ 4⅛″	1′ 3″	1′ 10″	1′ 8″	0.18	0.21
33″	5″	5″	2′ 2⅜″	2′ 4¼″	8¼″	2′ 6 15/16″	1′ 4½″	2′ 0″	1′ 9½″	0.19	0.23
36″	5″	5″	2′ 4¾″	2′ 6¾″	9 ″	2′ 9¾ ″	1′ 6″	2′2 ″	1′11″	0.22	0.26
39″	5″	5″	2′ 7¼″	2′ 9¼″	9¾″	3′ 0 9/16″	1′ 7½″	2′ 4″	2′ 0½″	0.25	0.29
42″	6″	6″	2′ 9⅝″	3′ 0 ″	10½″	3′ 3⅜ ″	1′ 9″	2′ 6″	2′ 3″	0.29	0.35
45″	6″	6″	3′ 0″	3′ 2½″	11¼″	3′ 6 3/16″	1′10½″	2′ 8″	2′ 4½″	0.33	0.40
48″	6″	6″	3′ 2⅜″	3′ 5¼″	1′ 0″	3′ 9″	2′ 0″	2′ 10″	2′ 6″	0.38	0.45
51″	6″	6″	3′ 4¾″	3′ 8 ″	1′ 0¾″	3′11 13/16″	2′ 1½″	3′ 0″	2′ 7½″	0.41	0.49
54″	6″	6″	3′ 7¼″	3′10¾″	1′ 1½″	4′ 2⅝″	2′ 3″	3′ 2″	2′ 9″	0.43	0.53
57″	6″	6″	3′ 9⅝″	4′ 1½″	1′ 2¼″	4′ 5 7/16″	2′ 4½″	3′ 4″	2′10½″	0.47	0.57
60″	6″	7″	4′ 0″	4′ 4 ″	1′ 3″	4′ 8¼ ″	2′ 6″	3′ 6″	3′ 0″	0.53	0.65
63″	6″	7″	4′ 2⅜″	4′ 6½″	1′ 3¾″	4′11 1/16″	2′ 7½″	3′ 8″	3′ 1½″	0.57	0.71
66″	6″	7″	4′ 4¾″	4′ 9¼″	1′ 4½″	5′ 1⅞″	2′ 9″	3′ 10″	3′ 3″	0.61	0.77
69″	6″	8″	4′ 7¼″	5′ 0 ″	1′ 5¼″	5′ 4 11/16″	2′10½″	4′ 0″	3′ 4½″	0.66	0.84
72″	6″	8″	4′ 9⅝″	5′ 2¾″	1′ 6 ″	5′ 7½″	3′ 0″	4′ 2″	3′ 6″	0.70	0.88

difficulty of obtaining with certainty a first-class wall of monolithic concrete of less thickness. The shape of the masonry invert is dependent upon the character of the excavation, whether it is in firm ground or soft ground, these being the terms applied to materials which would and would not stand when trimmed to the shape of the firm ground section.

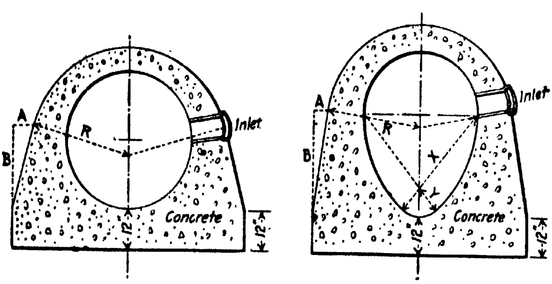


FIG. 149.—Standard plain concrete sections. (Bronx.)

For sewers of this type constructed on timber platforms or piles the line of the under side of the concrete invert should be horizontal. For reinforced-concrete sections, the thickness of masonry shown for the larger diameters may be somewhat reduced, according to J. H. Kimball,

formerly Designing Engineer, Commissioners of Sewerage of Louisville, Ky., to whom the authors are indebted for valuable assistance.

Borough of the Bronx.—Fig. 149 shows the standard forms of circular and egg-shaped sewers, constructed of unreinforced concrete, published in "Standard Details of Construction," 1913, Borough of the Bronx, N. Y., Richard H. Gillespie, Chief Eng. of Sewers and Highways. The minimum thickness of masonry, as given in these tables, is 6 in. for a minimum diameter of 33 in.

Table 131.—Standard Plain Concrete Sections, Borough of the Bronx, New York City

Circular	Crown	Width of base	Outside radius *R*	Offset *A*	Offset *B*	Concrete area, sq. ft
2′ 9″	6″	5′ 3″	2′ 1 1/2″	7 3/8	1′ 9 11/16″	11.94
3′ 0″	6″	5′ 6″	2′ 3″	7 1/4″	1′ 11 1/16″	12.82
3′ 3″	8″	6′ 3″	2′ 7 1/2″	7 5/16″	2′ 0 1/2″	16.41
3′ 6″	8″	6′ 6″	2′ 9	7 3/16″	2′ 1 7/8″	17.46
3′ 9″	8″	6′ 9″	2′ 10 1/2″	7 1/8″	2′ 3 1/4″	18.52
4′ 0″	8″	7′ 0″	3′ 0″	7 1/16″	2′ 4 5/8″	19.60

Egg shapes	Crown	Width of base	Outside radius *R*	Radius *X*	Radius *Y*	Offset *A*	Offset *B*	Concrete area sq. ft.
29″ × 40″	6″	4′ 9″	1′11 1/2″	2′ 10 1/8″	7 1/2″	5 7/16″	2′ 3 1/8″	12.82
32″ × 44″	6″	5′ 0″	2′ 1″	3′ 0 7/16″	7 1/2″	5 7/16″	2′ 5 1/2″	14.00
34″ × 46″	6″	5 3″	2′ 2″	3′ 2″	9″	6″	2′ 6 11/16″	14.78
38″ × 50″	8″	6′ 0″	2′ 7″	3′ 2 3/16″	9″	5 7/16″	2′ 8 3/16″	19.08
40″ × 53″	8″	6′ 3″	2′ 8″	3′ 4 11/16″	9″	6″	2′ 10 7/16″	20.33
42″ × 56″	8″	6′ 6″	2′ 9″	3′ 9 7/8″	12″	6 1/2″	3′ 0 3/4″	21.43

Gregory's Semi-elliptical Section.—The standard semi-elliptical section shown in Fig. 150, was worked out in 1910 by John H. Gregory in connection with the preparation of plans for a large trunk sewerage project. He stated that this section, which was designed to be built of concrete, is better adapted for sewers 6 ft. and over in diameter than for smaller ones. The several dimensions are given in terms of the diameter, D. The functions of the diameter were so chosen, starting with any diameter, D, in feet that with increments of 3 in. in the diameter, the resulting dimensions will come out in whole inches or inches and fractions of an inch in common use, as for example, quarters, eighths or sixteenths. Mr. Gregory further stated that the section is suitable for use only where the conditions are such that the side walls will be firmly supported by the sides of the trench. Where these conditions cannot be obtained, the side-wall sections should be modified to meet the conditions.

The horizontal and vertical diameters of the section are the same and the horizontal diameter is located one-third the length of the vertical diameter above the bottom of the sewer. The gross area of this section

on outside lines equals $1.2651D^2$, the area of the section inside = $0.8176D^2$ and the net area of masonry = $0.4475D^2$.

Table 132 shows the area of these sections and the net volume of masonry in cubic yards per linear foot for each size from 6 to 13 ft. 6 in.

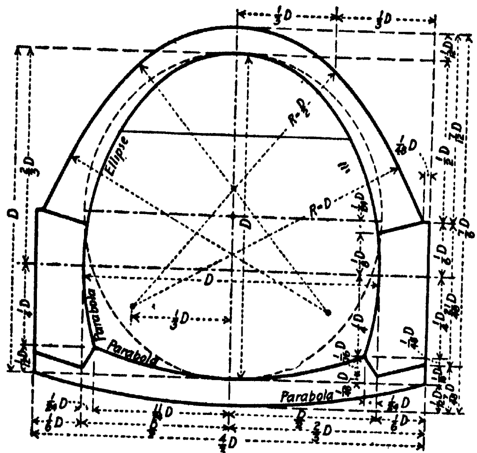


Fig. 150.—Gregory's standard semi-elliptical section.

TABLE 132.—AREA AND VOLUME OF MASONRY IN SEMI-ELLIPTICAL SEWERS
Gregory's Section (Fig. 150)

Inside diameter of sewer D (1)	Area in square feet: Gross area on outside lines $1.265D^2$ (2)	Area in square feet: Area of section inside $0.8176D^2$ (3)	Area in square feet: Net area of masonry $0.4475D^2$ (4)	Volume of masonry in cubic yards per linear foot $0.01657D^2$ (5)
6′ 0″	45.54	29.43	16.11	0.597
6′ 6″	53.45	34.54	18.91	0.700
7′ 0″	61.99	40.06	21.93	0.812
7′ 6″	71.16	45.99	25.17	0.932
8′ 0″	80.97	52.33	28.64	1.061
8′ 6″	91.40	59.07	32.33	1.197
9′ 0″	102.5	66.23	36.25	1.342
9′ 6″	114.2	73.79	40.39	1.496
10′ 0″	126.5	81.76	44.75	1.657
10′ 6″	139.5	90.14	49.34	1.827
11′ 0″	153.1	98.93	54.15	2.005
11′ 6″	167.3	108.1	59.18	2.192
12′ 0″	182.2	117.7	64.44	2.387
12′ 6″	197.7	127.7	69.92	2.590
13′ 0″	213.8	138.1	75.63	2.801
13′ 6″	230.6	149.0	81.56	3.021

in diameter by 6-in. steps. Additional data in regard to the hydraulic elements of this section are given in Table 128, Fig. 141, and the velocity and discharge for various diameters are shown in Fig. 146. Mr. Gregory further stated in *Engineering News*, March 12, 1914:

"In conclusion it should be pointed out that the dimensions given for the masonry section are a *minimum* and that not only would the best of materials and workmanship be required, but also careful inspection. Where these conditions cannot be obtained or where the sewers would be required

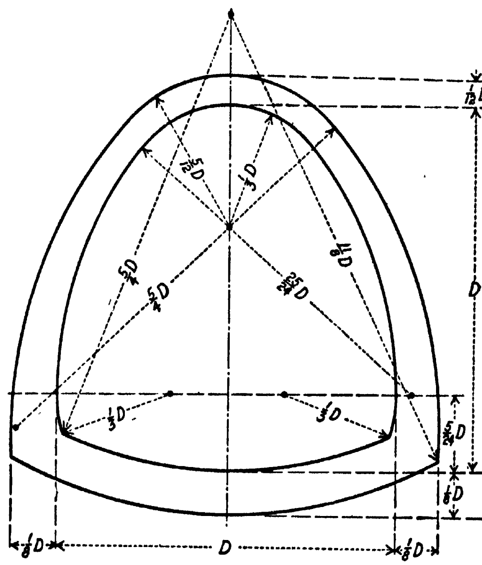


FIG. 151.—Authors' standard semi-elliptical section.

to carry heavy loads, the sections should be reinforced with steel or the dimensions increased, especially the arch and side walls."

Authors' Semi-elliptical Section.—The details of the semi-elliptical section shown in Fig. 151 and Table 133 were developed by the authors from the experience in constructing sewers of this type at Louisville, Ky. In all of the principal types, the stresses were carefully analyzed but no definite standards were developed in the Louisville work and on that account the sections actually constructed vary slightly from the section shown. This sewer is intended to be constructed of concrete reinforced with steel bars. Under favorable conditions the thicknesses of masonry shown may be slightly reduced while on the other hand for conditions of severe loading it may be desirable to increase them somewhat. For average conditions, however, the section shown is believed to be conservative.

TABLE 133.—DIMENSIONS OF AUTHORS' SEMI-ELLIPTICAL SEWER SECTION
(See Fig. 151)

1	2	3	4	5	6	7	8	9	10	11	12
			Thickness of concrete		Interior radii			Exterior radii			
Inside vertical diameter "D"	Area of waterway	Hydraulic mean radius	Crown	Center of invert and spring line	Crown intrados and side wall	Side intrados	Invert and side extrados	Crown extrados	Invert	Area of concrete	Quantity of concrete cu. yd. per lin. ft.
ft. in.	sq. ft.	ft.	ft. in.	ft. in	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	sq. ft.	
6 0	28.2	1.442	0 6	0 9	2 0	6 3	7 6	2 6	8 3	14.12	0.523
6 6	33.1	1.562	0 6½	0 9¾	2 2	6 9¼	8 1½	2 8½	8 11¼	16.58	0.614
7 0	38.4	1.683	0 7	0 10½	2 4	7 3½	8 9	2 11	9 7½	19.21	0.712
7 6	44.05	1.803	0 7½	0 11¼	2 6	7 9¾	9 4½	3 1½	10 3¾	22.08	0.817
8 0	50.1	1.923	0 8	1 0	2 8	8 4	10 0	3 4	11 0	25.10	0.930
8 6	56.6	2 043	0 8½	1 0¾	2 10	8 10¼	10 7½	3 6½	11 8¼	28.44	1.054
9 0	63.4	2.163	0 9	1 1½	3 0	9 4½	11 3	3 9	12 4½	31.80	1.177
9 6	70.7	2 284	0 9½	1 2¼	3 2	9 10¾	11 10½	3 11½	13 0¾	35.41	1.311
10 0	78.3	2.404	0 10	1 3	3 4	10 5	12 6	4 2	13 9	39.24	1.453
10 6	79.3	2 525	0 10½	1 3¾	3 6	10 11¼	13 1½	4 4½	14 5¼	43.26	1.602
11 0	94.75	2.646	0 11	1 4½	3 8	11 5½	13 9	4 7	15 1½	47.48	1.757
11 6	103.5	2 764	0 11½	1 5¼	3 10	11 11¾	14 4½	4 9½	15 9¾	51.89	1.921
12 0	112.75	2.884	1 0	1 6	4 0	12 6	15 0	5 0	16 6	56.51	2.092
12 6	122.4	3.005	1 0½	1 6¾	4 2	13 0¼	15 7½	5 2½	17 2¼	61.31	2.270
13 0	132 4	3.125	1 1	1 7½	4 4	13 6½	16 3	5 5	17 10½	66.32	2.456
13 6	142.7	3.245	1 1½	1 8¼	4 6	14 0¾	16 10½	5 7½	18 6¾	71.51	2.649
14 0	153 5	3.365	1 2	1 9	4 8	14 7	17 6	5 10	19 3	76 91	2.849

Area of waterway = $0.7831D^2$. Area of concrete section = $0.3924D^2$.

St. Louis Five-centered Arch.—The standard cross-section of the five-centered arch or semi-elliptical type of sewer shown in Fig. 152 was furnished by W. W. Horner, Principal Asst. Eng., St. Louis Sewer Department. Table 134 gives the leading dimensions and hydraulic properties of this section. The following notes in regard to the design of this standard section have been taken from a paper by P. J. Markmann, Office Eng., St. Louis Sewer Department.

In the preliminary studies, three systems of external forces were studied. The first, called the "standard" system, was composed of vertical forces due to the total weight of the backfill resting on the sewer arch and a small amount of horizontal earth pressure, depending in amount upon the angle of repose of the earth, assumed to be 25 deg. The second system of external forces consisted of vertical forces only, and ignored the existence of any horizontal earth pressure. This case would express the condition of the angle of repose approaching 90 deg., and would cover the possible case of horizontal forces in the "standard" system of loading having been assumed too great as compared with the vertical forces. The third system consisted of external forces acting normal to the center line of the arch, these forces being assumed equal to the weight of the fill, which is equivalent to a very wet condition or hydrostatic pressure. In each case analyses were made for varying

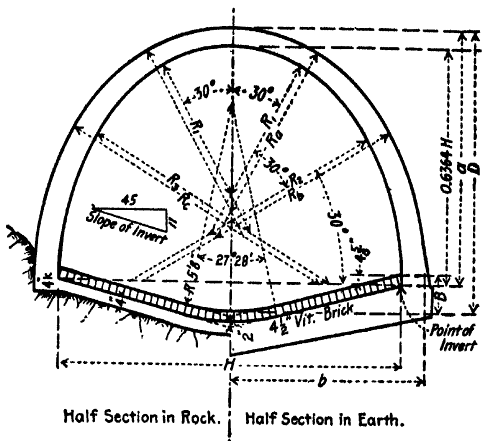


Fig. 152.—St. Louis five-centered arch sewer.

$R_1 = 0.4048H$; $R_2 = 0.5285H$; $R_3 = 0.7774H$; $Ra = 2.7321b - 0.8458(a + b)$; $R_b = 0.4650(a + b)$; $R_c = 1.8862(a + b) - 2.7321b$; $B = 0.1222H - 0.1667$; $D = 0.7586H - 0.5521$; Area $= 0.5609H^2 - 0.3854H - 0.1509$; Wetted perimeter $= 2.8211H - 0.8239$.

Table 134.—Properties of Five-centered Arch Sewer (Fig. 152)

H	R_1		R_2		R_3		B		D		Area	Wetted perim.	Hyd. rad.
ft.	ft.	in.	ft.	in.	ft.	in.	ft.	in.	ft.	in.	sq. ft.	ft.	ft.
6	2	$5\frac{1}{8}$	3	$2\frac{1}{16}$	4	8	0	$6\frac{13}{16}$	4	0	17.729	16.103	1.101
7	2	10	3	$8\frac{3}{8}$	5	$5\frac{5}{16}$	0	$8\frac{1}{4}$	4	$9\frac{1}{8}$	24.635	18.924	1.302
8	3	$2\frac{7}{8}$	4	$2\frac{3}{4}$	6	$2\frac{5}{8}$	0	$9\frac{3}{4}$	5	$6\frac{3}{16}$	32.664	21.745	1.502
9	3	$7\frac{3}{4}$	4	$9\frac{1}{16}$	6	$11\frac{15}{16}$	0	$11\frac{3}{16}$	6	$3\frac{5}{16}$	41.813	24.566	1.702
10	4	$0\frac{9}{16}$	5	$3\frac{7}{16}$	7	$9\frac{5}{16}$	1	$0\frac{11}{16}$	7	$0\frac{7}{16}$	52.085	27.387	1.902
11	4	$5\frac{7}{16}$	5	$9\frac{3}{4}$	8	$6\frac{5}{8}$	1	$2\frac{1}{8}$	7	$9\frac{1}{2}$	63.479	30.208	2.101
12	4	$10\frac{5}{16}$	6	$4\frac{1}{8}$	9	$3\frac{15}{16}$	1	$3\frac{5}{8}$	8	$6\frac{5}{8}$	75.994	33.029	2.301
13	5	$3\frac{1}{8}$	6	$10\frac{7}{16}$	10	$1\frac{1}{4}$	1	$5\frac{1}{16}$	9	$3\frac{3}{4}$	89.631	35.850	2.502
14	5	8	7	$4\frac{13}{16}$	10	$10\frac{5}{8}$	1	$6\frac{9}{16}$	10	$0\frac{13}{16}$	104.390	38.671	2.699
15	6	$0\frac{7}{8}$	7	$11\frac{1}{8}$	11	$7\frac{15}{16}$	1	8	10	$9\frac{15}{16}$	120.271	41.492	2.899
16	6	$5\frac{3}{4}$	8	$5\frac{1}{2}$	12	$5\frac{1}{4}$	1	$9\frac{7}{16}$	11	7	137.273	44.314	3.098
17	6	$10\frac{9}{16}$	8	$11\frac{13}{16}$	13	$2\frac{9}{16}$	1	$10\frac{15}{16}$	12	$4\frac{1}{8}$	155.397	47.135	3.297
18	7	$3\frac{7}{16}$	9	$6\frac{3}{16}$	13	$11\frac{15}{16}$	2	$0\frac{7}{8}$	13	$1\frac{1}{4}$	174.644	49.956	3.496
19	7	$8\frac{5}{16}$	10	$0\frac{1}{2}$	14	$9\frac{1}{4}$	2	$1\frac{7}{8}$	13	$10\frac{5}{16}$	195.011	52.777	3.695
20	8	$1\frac{1}{8}$	10	$6\frac{7}{8}$	15	$6\frac{9}{16}$	2	$3\frac{5}{16}$	14	$7\frac{7}{16}$	216.501	55.599	3.894
22	8	$10\frac{7}{8}$	11	$7\frac{9}{16}$	17	$1\frac{1}{4}$	2	$6\frac{1}{8}$	16	$1\frac{5}{8}$	262.845	61.240	4.292
24	9	$8\frac{9}{16}$	12	$8\frac{1}{4}$	18	$7\frac{7}{8}$	2	$9\frac{3}{16}$	17	$7\frac{7}{8}$	313.678	66.883	4.690
26	10	$6\frac{5}{16}$	13	$8\frac{15}{16}$	20	$2\frac{9}{16}$	3	$0\frac{1}{8}$	19	$2\frac{1}{16}$	368.996	72.525	5.088
28	11	4	14	$9\frac{5}{8}$	21	$9\frac{3}{16}$	3	$3\frac{1}{16}$	20	$8\frac{1}{4}$	428.804	78.167	5.486
30	12	$1\frac{3}{4}$	15	$10\frac{5}{16}$	23	$3\frac{7}{8}$	3	6	22	$2\frac{1}{2}$	493.098	83.810	5.884

depths of fill, 10 ft., 20 ft., 30 ft. and 40 ft. from the ground surface to the crown of the sewer.

The line of pressure in the arch for the standard system of forces was found to be a close approximation to an elliptical curve, and as the forces were assumed symmetrical, the major axis of this ellipse coincided with the vertical axis of the arch.

The arches were actually designed with a curvature following that of the line of pressure of the standard system of forces. The line of pressure for the second system of forces fell inside the standard line, thereby causing negative bending moments between the crown and springing line in the arch. The line of pressure for the third system of forces, for nearly all depths of fill, fell outside the standard line of pressure, causing positive bending moments between the crown and springing line.

TABLE 135.—PRINCIPAL DIMENSIONS OF SEWERS CONSTRUCTED IN LOUISVILLE, KY., 1907–1913

1	2	3	4	5	6	7	8	9
Contract number	Interior dimensions		Type of sewer	Thickness of concrete				Depth of fill over crown (ft.)
	Vertical diameter ft. in.	Horizontal diameter ft. in.		Crown (inches)	Invert (inches)	At springing line		
						Arch (inches)	Side wall (in.)	
2	15 2	15 6	Horse-shoe	11	12	17	17	25
	13 11	14 3	Horse-shoe	8	10	12½	12½	15
5	13 8	14 0	Horse-shoe	10	12	15	18⅝	30
56	12 9	13 0	Horse-shoe	8	10	14½	14½	20
14	12 3	12 3	Semi-elliptical	9	9	14	14	25
45	10 0	10 0	Semi-elliptical	8	8	16	16	42
6	10 1½	10 7	Horse-shoe	10	14	15	17¾	37
54	9 0	12 0	Horse-shoe	8	8	17	17	15.5
	9 0	9 0	Horse-shoe	8	8	15	15	15
62	7 6	10 0	Horse-shoe	8	8	12	12	13
55	8 3	8 3	Semi-elliptical	8	8	12	12	12
25	8 0	8 0	Semi-elliptical	9	7	15	15	15.5
	7 0	7 0	Semi-elliptical	8	7	13	13	13
48	6 3	6 3	Semi-elliptical	6	6	8¾	8¾	10
8	5 6½	5 10	Horse-shoe	6	6	7½	7½	17
	5 6	5 6	Circular	6½	6	10	10	12
43	4 11	5 2	Horse-shoe	7	6	10	10	18
16	4 6	4 6	Semi-elliptical	7	6	9	9	25
61	5 6	5 6	Circular	6	7	10	10	5
	3 3	3 3	Circular	5	8	8	8	10

Data from Contract drawings—Commissioners of Sewerage.

The sewer arch of any required size was designed of such varying thickness (increasing from crown to abutment) as to resist, in addition to the

direct thrust, not less than 50 per cent. of the moments indicated by the positions of the lines of pressure for each of the two extreme conditions of loading.

The hydraulic radius of this conduit is equal to the hydraulic radius of a circle whose diameter $d = 0.7422H$, where H is the horizontal diameter of the conduit. The area of the conduit is equal to the area of a circle whose diameter $d = 0.8H$. The hydraulic radius of the conduit is nearly 93 per cent. of that of a circle of equal area.

Louisville and St. Louis Sewers.—During 1907 to 1913, inclusive, there were constructed at Louisville, Ky., the main and intercepting sewers of a comprehensive system of sewerage. On this work J. H. Kimball was Designing Eng., J. B. F. Breed, Chief Eng., and Harrison P. Eddy, Consulting Eng. Practically all sewers were constructed of concrete, the majority of them being reinforced with steel bars. The sizes varied from small pipe sewers up to those 15 ft. and over in diameter. Table 135 gives the principal dimensions of a number of the larger sewers and is of interest in connection with Fig. 151, as showing the thicknesses of masonry actually constructed at Louisville. Additional data concerning these sewers will be found in other chapters of this book.

A considerable number of sewer sections of large size have been designed and constructed by the St. Louis Sewer Department, Tables 136–138. Two classes of concrete were used in the construction of these sections, according to *Engineering and Contracting*, Oct. 11, 1911. Mortar for Class A concrete had a ratio of 1 bbl. of cement to 7.6 cu. ft. of sand, and that for Class B concrete one of 1 bbl. of cement to 11.4 cu. ft. of sand. The concrete was made by mixing with the broken stone or gravel an amount of mortar of the proper class 10 per cent. in excess of the voids in the stone or gravel.

For Class A concrete the unit allowable stress in the concrete was assumed between 500 and 560 lb. per square inch and that for Class B concrete from 400 to 450 lb.

These arches were designed independently for particular conditions of live load, vibration and other conditions, including the personal equation of the designer. The Baden Public sewer arch is of Class A and the invert of Class B concrete. All other elliptical sections are of Class A concrete throughout. The River Des Peres horse-shoe sections are of Class A concrete throughout while all the other horse-shoe sections are of Class B concrete throughout.

Typical Sewer Sections.—A number of sewer sections are reproduced in Figs. 153 to 173 inclusive which are typical of different classes of structures designed to meet special conditions. All information is from official sources unless otherwise stated.

Fig. 153*a*.—Mass. Metropolitan Sewerage Comm., Neponset Valley Sewer, 1897, Wm. M. Brown, Jr., Chief Eng., 4 ft. 3 in. by 4 ft. 4½ in. Gothic section. Depth of cover approximately 18 ft. Material excavated was sand, gravel and clay.

Fig. 153*b*.—Mass. Metropolitan Sewerage Comm. Neponset Valley sewer, 1897, Wm.

TABLE 136.—CONCRETE SEWER ARCHES IN EARTH, ST. LOUIS

Horizontal diameter in ft.	Type	Depth of fill over crown, ft.	Thickness of concrete			Materials per lin. ft. of sewer			Where used
			Crown, in.	Springing line, in.	Invert, in.	Cu. yd. concrete	Cu. yd. vit. brick invert lining	Lb. steel	
33	Ellipt.	20	12	25	48	6.048	0.471	551.23	A
33	Ellipt.	30	18	29	63	6.201	0.471	591.00	A
33	Ellipt.	40	22	31	72	7.433	0.471	916.00	A
28	Ellipt.	10	9	18	28	3.409	0.398	320.00	A
28	Ellipt.	20	12	18	39	4.025	0.398	383.00	A
26	Ellipt.	10	9	16	25	3.162	0.370	313.00	A
26	Ellipt.	20	12	16	35	3.507	0.370	346.00	A
24½	Ellipt.	10	9	18	27	3.564	0.348	238.00	A
24½	Ellipt.	20	11	22	38	4.122	0.348	330.00	A
24½	Ellipt.	30	13⅝	24	48	4.558	0.348	332.00	A
23	Ellipt.	......	13	......	32	5.534	0.328	349.80	C
22½	Ellipt.	10	8	15	22	2.872	0.319	266.50	A
22½	Ellipt.	20	9	20	34	3.428	0.319	329.00	B
22½	Ellipt.	30	10	24	44	3.917	0.319	293.00	B
22	Horse-s.	10	11	18	25	3.649	0.320	437.50	B
22	Horse-s.	15	13	22	30	4.440	0.320	509.00	B
20	Horse-s.	15	12	21	28	3.811	0.291	447.00	B
20	Horse-s.	20	14	24	32	4.444	0.291	468.50	B
18	Horse-s.	15	11	19	26	3.130	0.262	369.00	B
18	Ellipt.	......	12	......	30	3.815	0.257	262.60	C
16	Horse-s.	10	9	15	20	2.189	0.233	307.00	B
16	Horse-s.	10	12	18	23	2.990	0.233	125.00	D
16	Horse-s.	20	15	24	30	3.810	0.233	185.50	D
16	Ellipt.	......	11	26	31	2.947	0.223	199.00	E
15½	Ellipt.	......	10	24	27	2.579	0.223	193.00	E
15½	Horse-s.	10	13	16	21	2.670	0.226	220.00	F
15½	Horse-s.	15	14	18	24	2.929	0.226	220.00	F
15	Ellipt.	10	7	......	14	1.561	0.214	117.00	E
15	Ellipt.	20	9	......	22¾	2.129	0.214	164.00	E
15	Ellipt.	30	10½	......	30½	2.662	0.214	218.50	E
14	Horse-s	10	12	18	23	2.617	0.203	134.00	D
14	Horse-s.	20	16	23	30	3.402	0.203	171.00	D
14	Horse-s.	10	10	15	22	2.093	0.203	236.50	B
13	Horse-s.	10	11	14	18	1.951	0.189	151.50	F
13	Horse-s.	15	12	15	20	2.116	0.189	192.50	F
13	Horse-s.	20	13	18	24	2.399	0.189	216.00	F
13	Ellipt.	10	7	......	12½	1.229	0.185	99.50	E
13	Ellipt.	20	8½	......	19½	1.593	0.185	136.50	E
13	Ellipt.	20	8	15	21½	1.671	0.182	113.13	A
12	Ellipt.	10	7	......	11¾	1.065	0.171	100.50	E
12	Ellipt.	20	8	15½	18½	1.462	0.171	111.50	E

TABLE 136.—CONCRETE SEWER ARCHES IN EARTH, ST. LOUIS (*Continued*)

Horizontal diameter in feet	Type	Depth of fill over crown, ft	Thickness of concrete			Materials per lin. ft. of sewer			Where used
			Crown, in.	Spring-ing line, in.	Invert, in.	Cu. yd. con-crete	Cu. yd. vit. brick invert lining	Lb. steel	
12	Horse-s.	10	10	16	20	1.920	0.174	110.00	D
12	Horse-s.	20	14	21	27	2.620	0.174	140.00	D
11	Horse-s.	10	10	14	18	1.630	0.160	87.00	D
11	Horse-s.	20	14	18	21	2.150	0.160	129.50	D
11	Horse-s.	10	9	12	16	1.408	0.160	123.50	F
11	Horse-s.	15	10	14	18	1.585	0.160	123.50	F
10	Horse-s.	15	9	12	17	1.297	0.145	131.50	F
10	Horse-s.	20	10	14	19	1.469	0.145	146.50	F
10	Horse-s.	10	10	14	18	1.507	0.145	74.50	D
10	Horse-s.	20	12	18	23	1.870	0 145	116.50	D

Note.—Locations indicated by letters in last column:
A. River Des Peres, Tunnel Line. D. South Harlem Joint.
B. River Des Peres, River Line. E. Glaise Creek Joint.
C. Baden Public, 1st Section. F. Rock Creek Joint.

TABLE 137.—CONCRETE SEWER ARCHES, ROCK BELOW POINT OF INVERT, ST. LOUIS

Horizontal diameter in ft.	Type	Depth of fill over crown ft	Thickness of concrete			Materials per lin. ft. of sewer			Where used
			Crown, in.	Spring-ingline, in.	Invert, in.	Cu. yd. con-crete	Cu. yd. vit. brick invert lining	Lb. steel	
33	Ellipt.	20	12	25	25	4.339	0.471	405.00	A
33	Ellipt.	30	18	29	29	5.136	0.471	450.00	A
33	Ellipt.	40	22	31	31	6.186	0.471	607.00	A
28	Ellipt.	10	9	18	18	2.357	0.398	243.00	A
28	Ellipt.	20	12	18	18	2.643	0.398	243.00	A
26	Ellipt.	10	9	16	16	2.029	0.370	258.50	A
26	Ellipt.	20	12	16	16	2.256	0.370	258.50	A
24½	Ellipt.	20	11	22	22	2.814	0.348	241.50	A
24½	Ellipt.	30	13⅝	24	24	3.285	0.348	217.21	A
23	Ellipt.	......	13	......	22	2.785	0.328	216.00	C
22½	Ellipt.	10	8	......	15	1.926	0.319	200.00	A
22½	Ellipt.	20	9	......	20	2.374	0.319	257.50	A
22½	Ellipt.	30	10	......	24	2.771	0.319	185.50	A
22	Horse-s.	10	11	18	25	2.970	0.320	200.00	B
22	Horse-s.	15	13	22	30	3.367	0.320	238.50	B
20	Horse-s.	15	12	21	28	2.925	0.291	197.00	B
20	Horse-s.	20	14	24	32	3.250	0.291	201.00	B
18	Horse-s.	15	11	19	26	2.437	0.233	120.50	B
18	Ellipt.	......	12	......	20	2.036	0.257	150.00	C

TABLE 137.—CONCRETE SEWER ARCHES, ROCK BELOW POINT OF INVERT, ST. LOUIS (*Continued*)

Horizontal diameter in ft.	Type	Depth of fill over crown ft	Thickness of concrete			Materials per lin. ft. of sewer			Where used
			Crown, in.	Springing line in.	Invert, in.	Cu. yd. concrete	Cu. yd. vit. brick invert lining	Lb. steel	
16	Horse-s.	10	9	15	20	1.833	0.233	120.50	B
16	Horse-s.	10	12	18	23	2.160	0.233	102.50	D
16	Horse-s.	20	15	24	30	2.720	0.233	143.50	D
15½	Horse-s.	10	13	16	21	2.041	0.226	141.50	F
15½	Horse-s.	15	14	18	24	2.222	0.226	141.50	F
14	Horse-s.	10	10	15	22	1.622	0.203	84.50	B
14	Horse-s.	10	12	18	23	1.952	0.203	112.50	D
14	Horse-s.	20	16	23	30	2.478	0.203	139.50	D
13	Horse-s.	10	11	14	18	1.505	0.189	94.50	F
13	Horse-s.	15	12	15	20	1.608	0.189	118.00	F
13	Horse-s.	20	13	18	24	1.891	0.189	134.20	F
13	Ellipt.	20	8	15	15	1.167	0.182	88.50	A
12	Horse-s.	10	10	16	20	1.542	0.174	89.50	D
12	Horse-s.	20	14	21	27	2.599	0.174	114.00	D
11	Horse-s.	10	10	14	18	1.360	0.160	71.00	D
11	Horse-s.	20	14	18	21	1.680	0.160	106.00	D
11	Horse-s.	10	9	12	16	1.099	0.160	80.00	F
11	Horse-s.	15	10	14	18	1.224	0.160	80.00	F
10	Horse-s.	15	9	12	17	1.017	0.145	77.50	F
10	Horse-s.	20	10	14	19	1.139	0.145	85.00	F
10	Horse-s.	10	10	14	18	1.137	0.145	64.00	D
10	Horse-s.	20	12	18	23	1.350	0.145	92.50	D

Note.—Locations indicated by letters in last column:

A. River Des Peres, Tunnel Line.
B. River Des Peres, River Line.
C. Baden public, 1st Section.
D. South Harlem Joint
E. Glaise Creek Joint.
F. Rock Creek Joint.

TABLE 138.—CONCRETE SEWER ARCHES, ROCK ABOVE POINT OF INVERT, ST. LOUIS.

Horizontal diameter in ft.	Type	Depth of fill over crown, ft.	Thickness of concrete			Materials per lin. ft. of sewer			Where used
			Crown, in.	Springing line in.	Invert, in.	Cu. yd. concrete	Cu. yd. vit. brick invert lining	Lb. steel	
22	Horse-s.	10	11	18	18	2.782	0.320	200.00	B
22	Horse-s.	15	13	22	22	3.160	0.320	238.50	B
16	Horse-s.	10	12	18	18	1.991	0.230	100.50	D
16	Horse-s.	20	15	24	24	2.809	0.230	140.50	D
15½	Horse-s.	10	13	16	16	1.939	0.226	141.50	F
15½	Horse-s.	15	14	18	18	2.100	0.226	141.50	F
14½	Horse-s.	10	12	15	15	1.727	0.211	129.50	F

TABLE 138.—CONCRETE SEWER ARCHES, ROCK ABOVE POINT OF INVERT, ST. LOUIS (*Continued*)

Horizontal diameter in ft.	Type	Depth of fill over crown, ft.	Thickness of concrete			Materials per lin. ft. of sewer			Where used
			Crown, in.	Springing line in.	Invert, in.	Cu. yd. concrete	Cu. yd. vit. brick invert lining	Lb. steel	
14	Horse-s.	10	12	18	18	1.663	0.203	80.50	D
14	Horse-s.	20	16	23	23	2.075	0.203	137.00	D
13	Horse-s.	10	11	14	14	1.435	0.189	94.50	F
13	Horse-s.	15	12	15	15	1.521	0.189	118.00	F
13	Horse-s.	20	13	18	18	1.777	0.189	134.00	F
12	Horse-s.	10	10	16	16	1.286	0.174	89.00	D
11	Horse-s.	15	10	14	14	1.164	0.160	80.50	F
11	Horse-s.	10	9	12	12	1.039	0.160	80.50	F
10	Horse-s.	15	9	12	12	0.945	0.145	77.50	F
10	Horse-s.	20	10	14	14	1.067	0.145	89.00	F

Note.—Locations indicated by letters in last column:

A. River Des Peres, Tunnel Line. D. South Harlem Joint.
B. River Des Peres, River Line. E. Glaise Creek Joint.
C. Baden Public, 1st Section. F. Rock Creek Joint.

M. Brown, Jr., Chief Eng., 4 ft. by 4 ft. 1½ in. Gothic section. Left half of figure construction for rock tunnel; right half construction for tunnel in hard gravelly soil.

Fig. 153*c*.—Philadelphia, Pa., 1906, Geo. S. Webster, Chief Eng., 4 ft. 9 in. standard circular sewer. The left half shows minimum section; right half construction in "reduced" cradle. Steel reinforcing over piles equal to 3/4-in. square bars 12 in. c. to c. Piles, 12-in. yellow pine 3 ft. apart both ways.

Fig. 153*d*.—Philadelphia, Pa., standard sewer section, 1906, Geo. S. Webster, Chief Eng. 4 ft. 9 in. circular sewer. Right half of section, construction in "maximum cradle," on piles 3 ft. 6 in. c. to c. transversely, and 3 ft. c. to c. longitudinally. Steel reinforcing over piles equal to 3/4 in. square bars 12 in. c. to c. Left half of section, construction on platform and piles. Platform of 6-in. yellow pine planking on 8 by 8 in. yellow pine stringers 3 ft. apart longitudinally. Piles 12-in. yellow pine, 3 ft. apart longitudinally and 3 ft. 9 in. c. to c. transversely.

Fig. 153*e*.—Truro, Nova Scotia, 1902, Lea & Coffin, Eng., 27-in. circular sewer, monolithic concrete to springing line of brick arch. Concrete used because of cheapness under given conditions as compared with brickwork. *Eng. Rec.*, Aug. 30, 1902.

Fig. 153*f*.—Philadelphia, Pa., Magee St. sewer, 1900, Geo. S. Webster, Chief Eng. 9 ft. 6 in. circular. Left half of section, construction in earth cut on piles with earth cover. Platform yellow pine, 6-in. planks on 8 × 8-in. caps with 12-in. piles set 3 ft. c. to c. in each direction. Right half of the section, construction in rock cut.

Fig. 154*a*.—Borough of Brooklyn, New York City, Gold St. Relief Sewer, 1907, E. J. Fort, Chief Eng., 13 ft. 6 in. circular section. The figure shows two methods of construction. In a third method the section was built entirely of concrete with 16 in. thickness at the crown and 20 in. at the springing line. A fourth type had a segmental concrete arch and concrete foundation of the same general dimensions as the left half of the section shown. Platform on earth, 2-in. plank laid on 4-in. sills; platform on piles, constructed of 6 in. plank floor laid on 10 × 12-in. capping on 12-in. spruce or pine piles, spaced 3 ft. 9 in. c. to c.

Fig. 154*b*.—Borough of Queens, New York.—Trunk sewer in Myrtle and St. Nicholas Aves., 1907, J. H. Johnson, Chief Eng., 15 ft. circular section. Depth of cover about 15 ft.; excavation in dry sandy soil. Reinforced with Johnson corrugated bars "new style;" extrados bars 7/8 in., 12 in. c. to c., transverse intrados bars, 1 in., 12 in. c. to c.; longitudinal bars, 3/4 in., spaced as shown. Sections under 15 ft. of the same general form; 11-ft. 3-in. section 12 in. thick at crown and 27 in. at springing line. Reinforcement as in 15-ft. section except outside transverse rods were 3/4 instead of 7/8 in. Sections under 4½ ft. had 3/4-in. transverse rods, 12 in. on centers and three 3/4-in. longitudinal rods over crown and

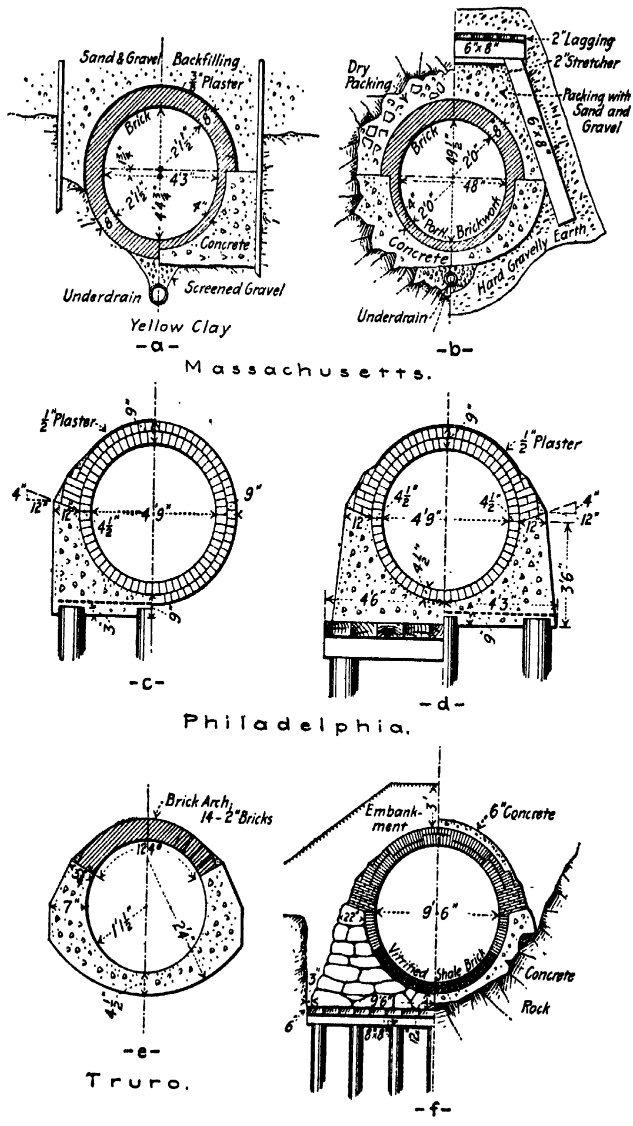


FIG. 153.—Typical circular sections with brick arches.

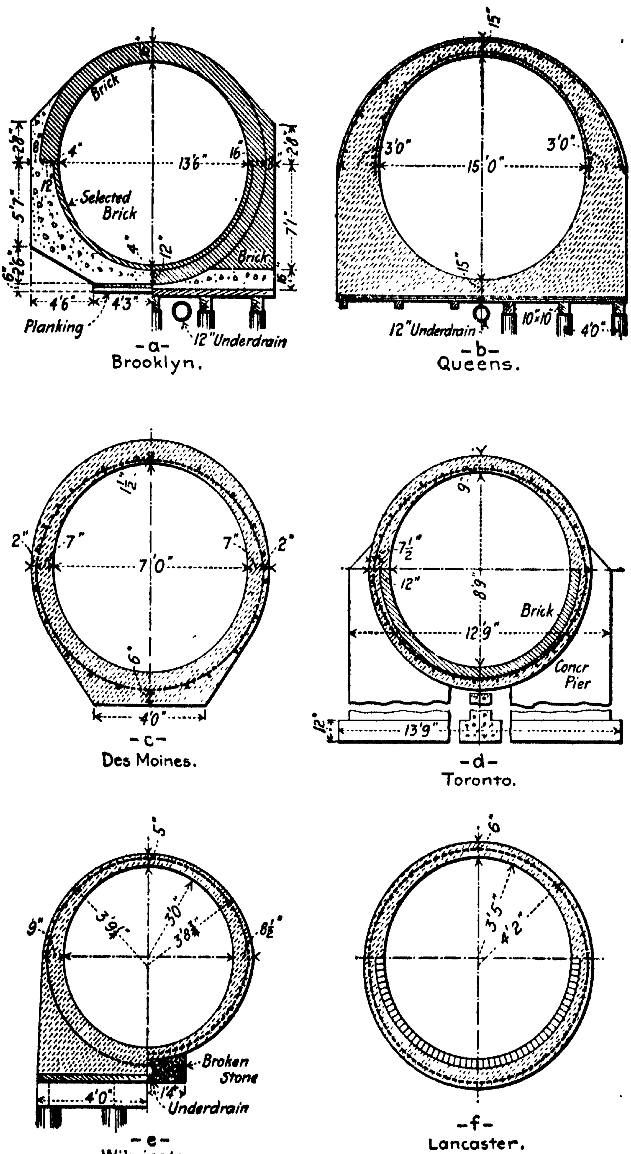


FIG. 154.—Typical circular sections.

haunches. Arches in all sections 6 in. thick at the crown and 6 to 9 in. at springing line. Sizes from 4½ to 5½ ft. diameter, inclusive, reinforced with 3/4-in. lateral rods 12 in. on centers and 3/4-in. longitudinal rods 18 in. on centers. Arch rings 6 to 8 in. thick at crown and 12 to 15 in. at the springing line. The 9-1/2 and 10-ft. sections were similar to those just described. The thickness at crown and springing line of 9 1/2 and 10-ft. sections was 12 and 24 in. respectively for both sizes. At one point, cover over 9-1/2-ft. sewer 22 ft. deep; sizes from 5½ ft. down had 8 to 10 ft. of cover. Practically no water encountered. Reference, *Eng. Rec.*, vol. lvi, p. 599.

Fig. 154*c*.—Des Moines, Iowa, Ingersoll Run Sewer, 1905, John W. Budd, City Eng., 7-ft. circular sewer, with 1/2-in. transverse bars 12 in. c. to c. and 1/4-in. longitudinal bars spaced as shown. *Eng. Rec.*, April 28, 1906.

Fig. 154*d*.—Toronto, Canada, High-Level intercepting sewer, 1910, Charles H. Rust, City Eng. Circular reinforced concrete sewer on concrete piers crossing filled ground. Transverse bars, 5/8-in. on 4-in. centers; longitudinal bars, 5/8-in. rods 12 in. c. to c. Lining below springing line, vitrified brick. In trench, section was plain concrete with vitrified brick invert lining. Thickness at crown was 12 in.; at springing line, 17½ in.; at invert, 12 in.;

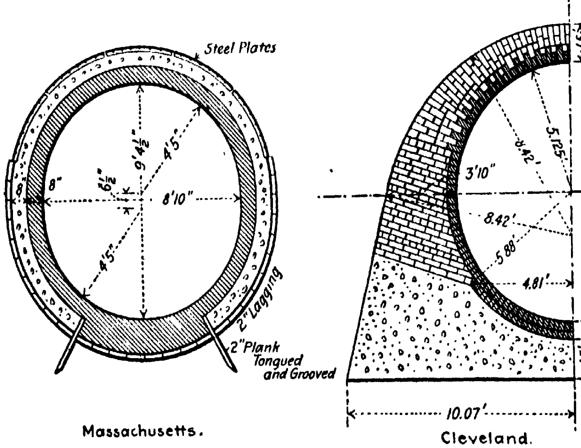


FIG. 155.—Typical circular sections.

invert below brick lining, 7½ in.; maximum width of plain concrete section 11 ft. 8 in.; concrete invert has horizontal base 3 ft. 6 in. wide and its sides slope upward 2 ft. 5 in. vertically in a horizontal distance of 4 ft. 1 in. *Eng. Rec.*, March 18, 1911, p. 304.

Fig. 154*e*.—Wilmington, Del. Price's Run sewer, 1903, T. Chalkley Hatton, Consulting Eng., 6 ft. circular section. Left half for shallow cut where sewer was largely above ground; used with and without platform. Right half, construction entirely below ground. With a thickness of only 5 in. at the crown, the sections withstood without fracture all load they will be subjected to at any time. Reinforcement, woven wire fabric of No. 8 wire with No. 6 wire selvage and 6 × 4-in. mesh. A 6½ ft. sewer of same type with same thicknesses was constructed, but a 9-ft. 3-in. section had a crown thickness of 8 in., 12 in. at the springing line and 8 in. of concrete at invert in section like right half of figure. Several hundred feet of this 9-ft. 3-in. section were built on pine piles 36 to 8 ft. long, four piles to each bent, spaced 3 ft. 10½ in. centers, and bents 4 ft. between centers. Each bent had a 10 × 12-in. yellow pine cap carrying floor of 3 × 12-in. hemlock. Reinforcement, expanded steel, 6-in. mesh, No. 6 gage, approximately 2 in. from the inner surface. (*Eng. Rec.*, May 21, 1904.)

Fig. 154*f*.—Lancaster, Pa., 1903, Samuel M. Gray, Eng., 6-ft. 10-in. circular sewer reinforced with 3-in. No. 10 expanded metal and inside below springing line lined with hard burned or vitrified brick. Alternative design had concrete foundation and brick arch;

greater roughness estimated to require 2 in. more diameter, giving 38.48 sq. ft. as against 36.67 sq. ft. for concrete sewer; had three rings brickwork on concrete base 9 ft. wide, 6 in. thick below brick lining of invert and extending vertically on sides to springing line. Alternate section required 16.24 cu. ft. brick and 16.48 cu. ft. concrete per linear foot; quantities for concrete sewer illustrated were 13.79 cu. ft. concrete and 4.08 cu. ft. brickwork. Sewer constructed as illustrated on account of the greater comparative economy.

Fig. 155*a*.—Mass. Metropolitan Sewerage Comm., North Metropolitan System, 1893, Howard A. Carson, Chief Eng., 8 ft. 10 in. × 9 ft. 4-1/2 in. Gothic section; built in pneumatic tunnel in soft clay underlaid by very wet sand. As an indication of the extent of groundwater, at one point it was impossible even with five compressors running to excavate nearer than 1½ ft. to grade of bottom of masonry until following method was used. Work started as low as possible and concrete lining used for sides, roof and heading. The 2-in. tongued and grooved planks set radially, prevented wet sand flowing in at the bottom. Upper part of arch secured by 1/8-in. by 1-ft. by 3-ft. curved steel plates, bolted to each other and supported by 8 × 8-in. temporary posts. With concrete lining in place, it was possible to hold air pressure and allow remainder of excavation to be made and brick invert and linings set. Sides and bottom of section held in place by 2-in. plank lagging. Found later that with arch built first, same results were obtainable without use of concrete; brick arch built on wall plates, these and arch supported by braces from axial beam; invert then built up to wall plates, and the space left by removal of wall plates filled with brick. Brick arches always 12 in. thick. *Eng. News*, Feb. 8, 1894.

Fig. 155*b*.—Cleveland, Ohio, Walworth Sewer, 1898, 10 ft. 3 in. circular section, very heavy on account of yielding plastic blue clay, unable to carry more than 2 tons per square foot. Thickness of arch increased gradually from crown to springing line, and arch bricks arranged in alternate headers and stretchers in Flemish bond. To avoid excessively thick mortar joints masonry was broken up as shown. Entire arch cut into segments separated by cylindrical surfaces and radial planes. Inner and outer faces of brick parallel with inner surface of completed sewer. Number of courses to build any particular cylindrical segment one more than the number in next inner segment, and one less than number in next outer segment; mortar joints of ordinary thickness were thus obtained in all portions of arch. Surfaces separating inner and outer rings of segments, as well as extrados of arch plastered with Portland cement mortar. Radial thickness of each part of superimposed masonry segments adjusted to break joints in adjoining segments by at least 4 in.

Sewer built on 3-in. oak plank, laid across sewer line on 3 × 12-in. oak sleepers, not more than 4 ft. c. to c. bedded in clay. Entire lower portion natural cement concrete. Top of concrete brought to plane inclining downward and inward 4 horizontal to 1 vertical. Minimum thickness of concrete under two rings of lining brick of the invert, 1.5 ft. for sewers from 8 ft. to 14 ft. 9 in. inclusive, and 2 ft. for larger sizes. Side walls brick laid in English bond in natural cement mortar, carried upward from concrete with courses pitching inward parallel to the upper surface of the concrete. Two concentric rings of brickwork in invert lining instead of one, in order to obtain a smoother inner surface.

Crown thickness fixed arbitrarily according to conditions in each case and thickness at springing line determined by formula given earlier in Chapter XI. Thickness at any other

TABLE 139.—PRINCIPAL DIMENSIONS OF WALWORTH SEWER, CLEVELAND, OHIO

Diameter, ft. in.	Thickness of masonry in feet. Crown	On horiz. line through center of sewer	Center of invert	Width of concrete foundation	Diameter, ft. in.	Thickness of masonry in feet Crown	On horiz. line through center of sewer	Center of invert	Width of concrete foundation
8 0	1.1	2.54	2.25	16.20	12–3	1.5	3.554	2.25	23.54
8 6	1.1	2.67	2.25	17.08	13–6	1.8	3.82	2.25	25.64
9 6	1.5	2.92	2.25	18.84	14–9	1.8	4.07	2.25	27.70
9 9	1.5	2.98	2.25	19.26	15–0	1.8	4.11	2.75	28.36
10 3	1.5	3.10	2.25	20.14	15–9	1.8	4.26	2.75	29.58
11 6	1.5	3.38	2.25	22.26	16–6	1.8	4.39	2.75	30.78

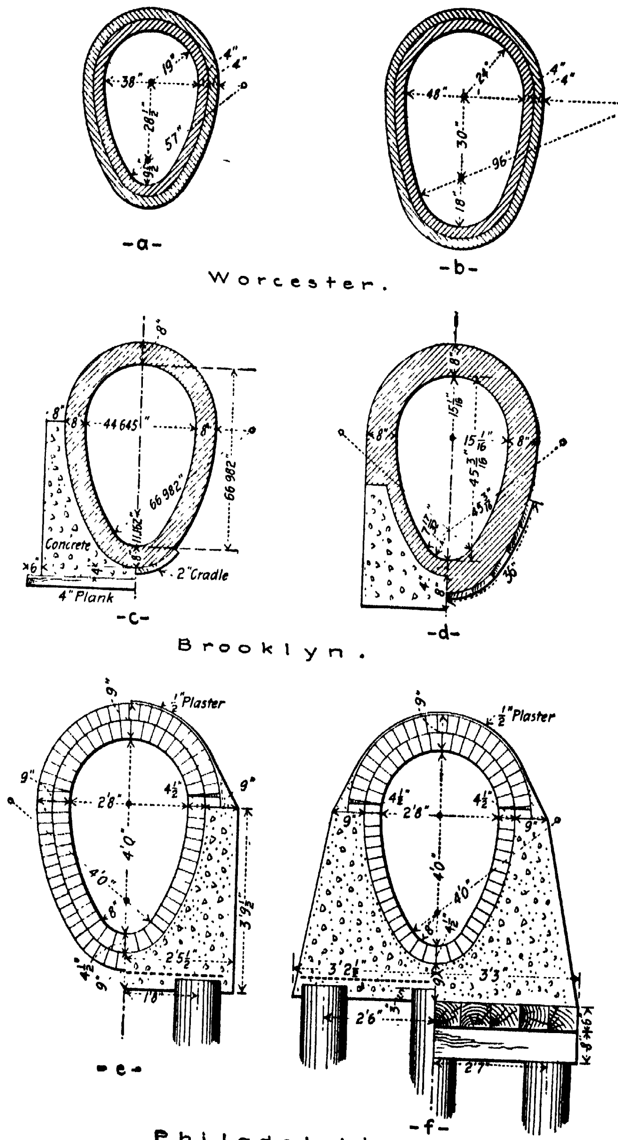


FIG. 156.—Typical egg-shaped sections.

point of arch determined by drawing arc of circle through these two points, this arc having its center directly below the center of the sewer. Below springing line, wall had batter of 1 horizontal to 4 vertical.

Other sections of various diameters were constructed along general plan of section shown; principal dimensions of several are given in Table 139. The invert in each case was lined with two rings of brick, a total of about 9 in. in thickness.

These sections are noteworthy for heavy masonry to retain line of resistance within middle third of section at all points and to spread thrust on soil to reduce soil pressure to not more than 2 tons per square foot. Sections also noteworthy on account of construction of arch and the unusual bonding of the brickwork adopted as productive of a much more stable structure than would result from use of ordinary bond. *Trans. Am. Soc. C. E.*, December, 1905.

Fig. 156*a*.—Worcester, Mass., Sewer Dept., 38 × 50-in. brick, egg-shaped sewer, typical of construction used extensively in many old systems throughout the country. In recent years, however, this type has been replaced largely by sections shown in Figs. 156c, d, e and f. Many of these old sewers show but few signs of distortion due to earth pressures. Where this type was built on steep grades in combined systems the invert bricks have been worn to a considerable extent and in some cases worn through, causing backfilling and supporting earth outside of brickwork to be washed away and resulting in caving in of sewer. This trouble overcome by making invert masonry heavier and lining invert with hard-burned or vitrified brick, calculated to resist wear better.

Fig. 156*b*.—Worcester, Mass., Sewer Dept., 48 × 72-in. brick, egg-shaped sewer, interesting on account of special shape used in several instances in that city.

Fig. 156*c*.—Borough of Brooklyn, New York City, 1901, H. R. Asserson, Chief Eng., 54-in. brick, egg-shaped sewer, with two types of construction. This sewer was designated by the size of the equivalent circular sewer instead of by dimensions of the egg-shaped section.

Fig. 156*d*.—Borough of Brooklyn, New York City, Bureau of Sewers, 1913, E. J. Fort, Chief Eng., standard 36-in. egg-shaped sewer of much interest when compared with Fig. 156e.

Fig. 156*e*.—Philadelphia, Pa., 1906, Standard sections, Geo. S. Webster, Chief Eng., 2-ft. 8-in. × 4-ft. egg-shaped sewer. Left half, construction in firm material when minimum section can be used; right half, construction called "reduced" cradle. Reinforcing bars over piles equal in area to 3/4 in. square bars, 12-in. centers. Piles 12-in. yellow pine 3 ft. apart longitudinally and 3 ft. 4 in. transversely.

Fig. 156*f*.—Philadelphia, Pa., Standard sections, 1906, Geo. S. Webster, Chief Eng., 2-ft. 8-in. × 4-ft. egg-shaped sewer. Left half, construction in "maximum" cradle on piles, reinforcing bars over piles equivalent to 3/4 in. square bars 12-in. centers, piles 12 in. in diameter spaced 3 ft. longitudinally and 2 ft. 6 in. transversely. Right half, construction on timber platform and piles; platform 6-in. yellow pine planking on 8 × 8-in. yellow pine stringers on 12-in. yellow pine piles 3 ft. apart longitudinally and 2 ft. 7 in. apart transversely. Where sewers are on steep grades, inside below springing line has one ring of vitrified shale brick.

Fig. 157*a*.—Worcester, Mass., Sewer Dept., 1899, H. P. Eddy, Supt., Water St. 40 × 54 in. inverted egg-shape intercepter. Average depth to crown of sewer, 17 ft. Sewer constructed in tunnel, largely rock but partly earth roof requiring bracing. Section chosen for its economy of space with wooden timbering and the additional inside head room available.

Fig. 157*b*.—Mass. Metropolitan Sewerage Comm., 1891, Howard A. Carson, Chief Eng. Sec. 1, North Metropolitan Outfall Sewer, Deer Island near pumping station. Catenary 6 × 6½-ft. type. Average depth of cover about 8 ft. Section designed to act under slight head; brick arch made extra heavy to produce excess of downward pressure. *Eng. News*, Feb. 8, 1894.

Figs. 157*c*, *d*, *e* and *f*.—Massachusetts Metropolitan Sewerage Comm., North Metropolitan Sewer, Section No. 26, 1892, Howard A. Carson, Chief Eng. Catenary 5¾ × 6¼-ft. section. Conditions generally permitted building invert in excavation without special foundation. Nearly half distance was in clay permitting an all-brick section but balance was in clay, sand or gravel requiring various forms shown. The entire length was protected by a timber platform with clay or concrete backfill between platform and sewer, except about 300 ft. where section 157e right half was used. Average depth of fill for sections in open cut, about 17 ft. Average depth above crown of sewer to surface of ground for tunnel section, about 24 ft. *Eng. News*, Feb. 8, 1894.

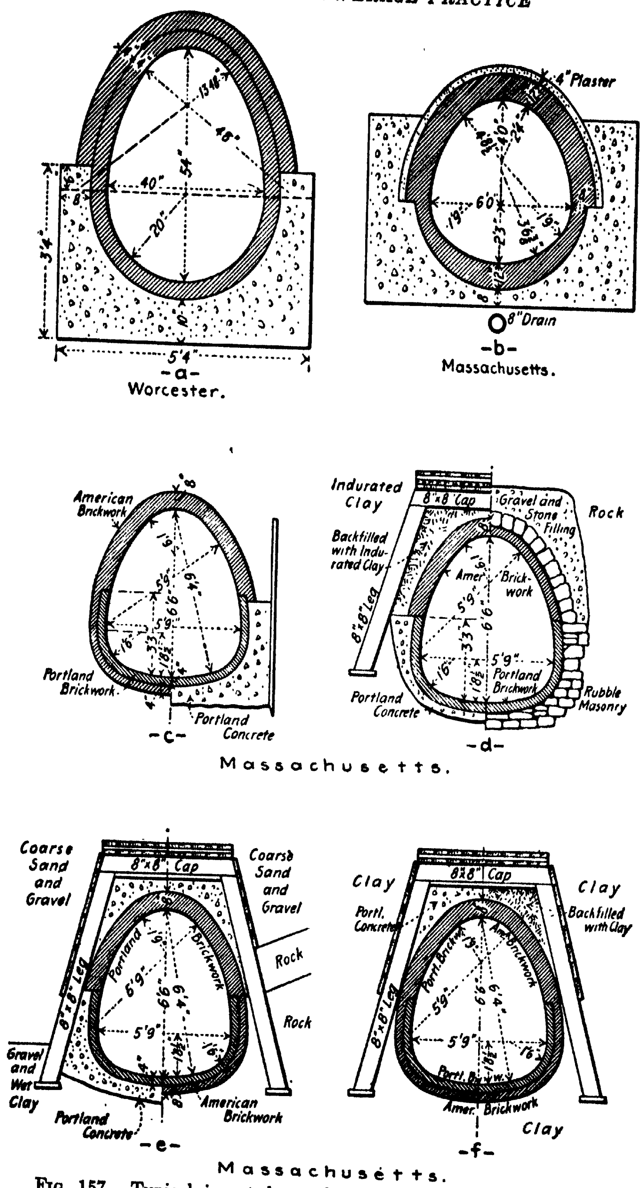


Fig. 157.—Typical inverted egg-shaped and catenary sections.

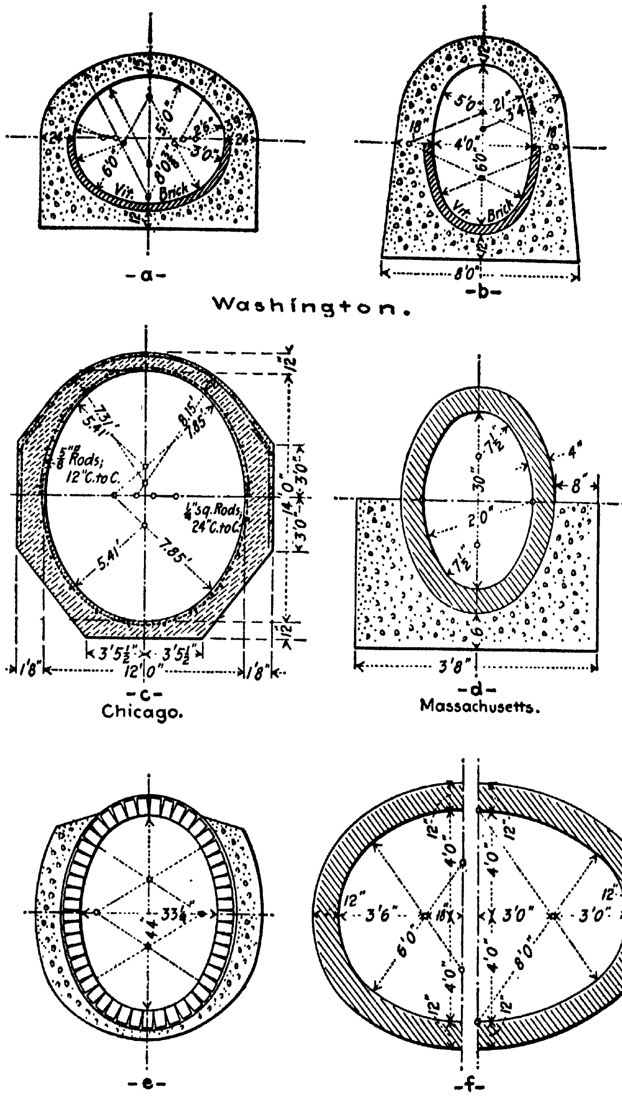


Fig. 158.—Typical elliptical sections.

Fig. 158*a*.—Washington, D. C., main conduit near pumping station, 1905, designed in office of Engineer Commissioner, District of Columbia. Oval 9 × 7-ft. 2-in. sewer. Short length of section connects main 6 × 6-ft. horse-shoe sewer with trunk sewer, and discharges into cunette in section shown in Fig. 168*c*.

Fig. 158*b*.—Washington, D. C., low area trunk sewer, 1905, designed in office of Engineer Commissioner, District of Columbia. Oval 4 × 6-ft. sewer. About 100 ft. built. This section and Fig. 158*a* selected to fill special requirements.

Fig. 158*c*.—Chicago, Ill., Western Ave. sewer, 1910, Isham Randolph, Chief Eng., Sanitary District of Chicago. Elliptical 12 × 14-ft. sewer. Excavation generally in stiff blue clay, average cover, 10 ft. Inside transverse bars, 5/8 in. square, 12 in. c. to c.; transverse extrados bars, 5/8 in. square, 12 in. c. to c.; longitudinal bars, 1/4 in. square 24 in. c. to c. Reinforcement used in but few places. Under Illinois & Michigan Canal, section changed to 12 × 9-ft. ellipse for distance of 60 ft. long with 5-ft. fall in that length. *Engineering and Contracting*, May 4, 1910, Feb. 11, 1914.

Fig. 158*d*.—Mass. Metropolitan Sewerage Comm., North Metropolitan Sewer, Section 41, 1892, Howard A. Carson, Chief Eng. Elliptical sewer, 1 ft. 8 in. × 2 ft. 6 in. Average cover, about 10 ft. Excavation in sand, gravel, ledge, boulders, filling and very fine sand containing much water. In places the fine sand was removed to 1 ft. below bottom of sewer and replaced with gravel. In other places, piles averaging 25 ft. were driven, bents 2 ft. on centers, with 8 × 10-in. caps and 2-in. flooring. Ledge was replaced by tamped gravel for 6 in. below bottom of brickwork. In sand, excavation carried to firm foundation and the brickwork bedded in and surrounded by gravel. In fine running sand, sewer laid in cradle of 1-in. boards on 2 × 4-in. ribs, and cradle covered with broken stone. Cradle lined with tar paper. Another section had cradle of two thicknesses of boards with tar paper between. *Eng. News*, Feb. 8, 1894.

Fig. 158*e*.—Altoona, Pa., 1896. Oval sewer, 33-1/4 × 44-in. Section had one-ring brickwork and 4 to 8 in. concrete, with invert of vitrified shale paving brick. Cost claimed to be less than cost of two-ring brick sewer. Proc. Engr. Club of Philadelphia, 1897, vol. xiv, page 91.

Fig. 158*f*.—Richmond, Va., 1912. False elliptical 8 × 10-ft. and 8 × 12-ft. sections, chosen on account of insufficient depth for circular sewer. Curves of arch and invert were three-centered, with row of headers at point of change of radius to tie the rings together. On account of shallow cover buttresses were built every 12 ft. to give arch good bearing against sides of ditch. Double-track railroad crosses sewer with only about 4-ft. cover. Portion constructed in 4 to 8-ft. rock cut, where concrete invert lined with 1 ring of brick and arch of 3 rings of brick were used. Fig. 158*f* shows one-half of each of the two sizes. *Engineering and Contracting*, Nov. 20, 1912.

Fig. 159*a*.—Mass. Metropolitan Sewerage Comm., North Metropolitan Sewer, Section 14, 1892, Howard A. Carson, Chief Eng. Basket-handle section, 8 ft. 2 in by 8 ft. 10 in. Left half of section, construction in firm material where bottom could be shaped to invert; right half, construction on timber platform on piles. Platform was 4-in. plank floor on 10 × 12-in. caps, on piles spaced 2 ft. 7 in. centers transversely. *Eng News*, Feb. 8, 1894.

Fig. 159*b*.—Mass. Metropolitan Sewerage Comm., North Metropolitan Sewer, Contract Section 14, 1892. Howard A. Carson, Chief Eng. Basket-handle sewer, 8 ft 4 in. × 9 ft. 2 1/2 in. used where material below springing line was sand and gravel and that above was clay. Sewer arch backfilled with gravel. *Eng. News*, Feb. 8, 1894.

Fig. 159*c*.—Washington, D C., Outfall Sewer, 1904, designed in office of Engineer Commissioner of District of Columbia. Basket-handle section, 9 ft. 4 in. × 8 ft. 4 in. Left half construction in firm ground; right half, construction in yielding soil or in insecure ground. Several hundred feet on piles, masonry section same as right half of figure. Pile spacing, one in center, one on either side 3 ft. 7-1/8 in. from center, and one outside pile on each side 3 ft. 4 in. from center of next adjacent pile, making five piles to bent, bents spaced 3 ft. 6 in. c. to c. Another section built on 3-in. yellow pine floor on 10 × 12-in. yellow pine caps on bents containing six piles, spaced 2 ft. 8 in. on centers.

Fig. 159*d*.—Pittsburgh, Pa., Try St. drainage sewer, Bureau of Surveys, Charles M. Reppert, Div. Eng. Basket-handle section 7 ft. 4 in. × 7 ft. 9-1/2 in. Left half, construction for firm ground; right half, construction for soft foundation. In latter case 1/2-in. bars, 6 in. c. to c. were placed in invert. A 6-ft. 8-in., × 7-ft. 1/2-in. section was also constructed, 9 in. thick at crown and 18 in. at springing line for firm-ground section and 30 in. for soft-ground section; and invert below vitrified shale brick lining, 8 in. thick. Maximum width of

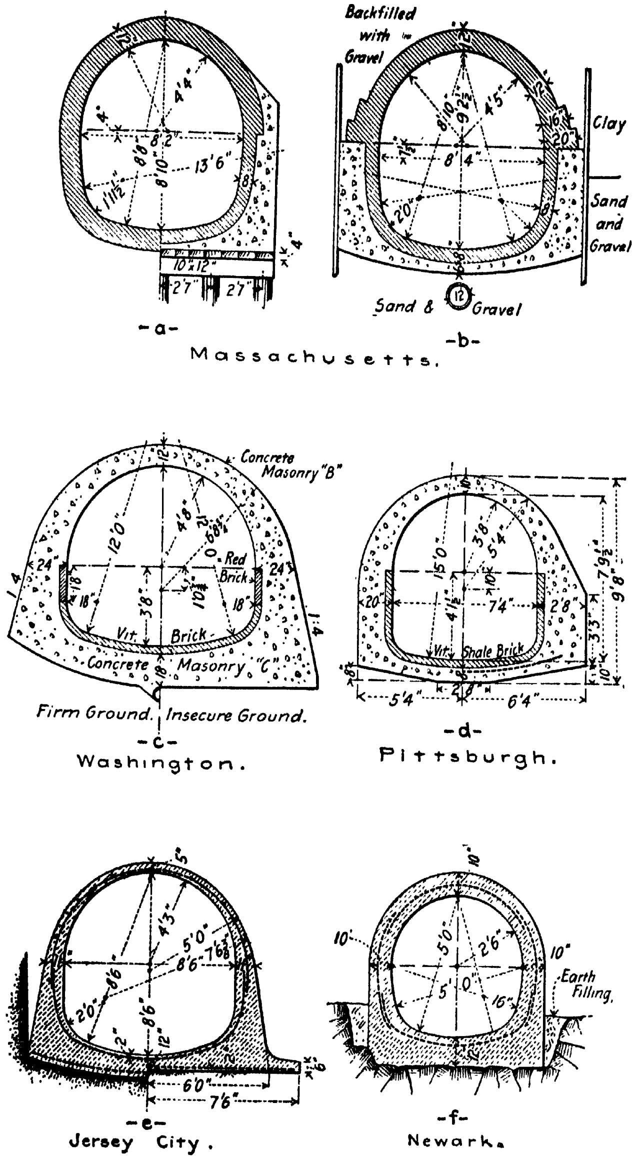
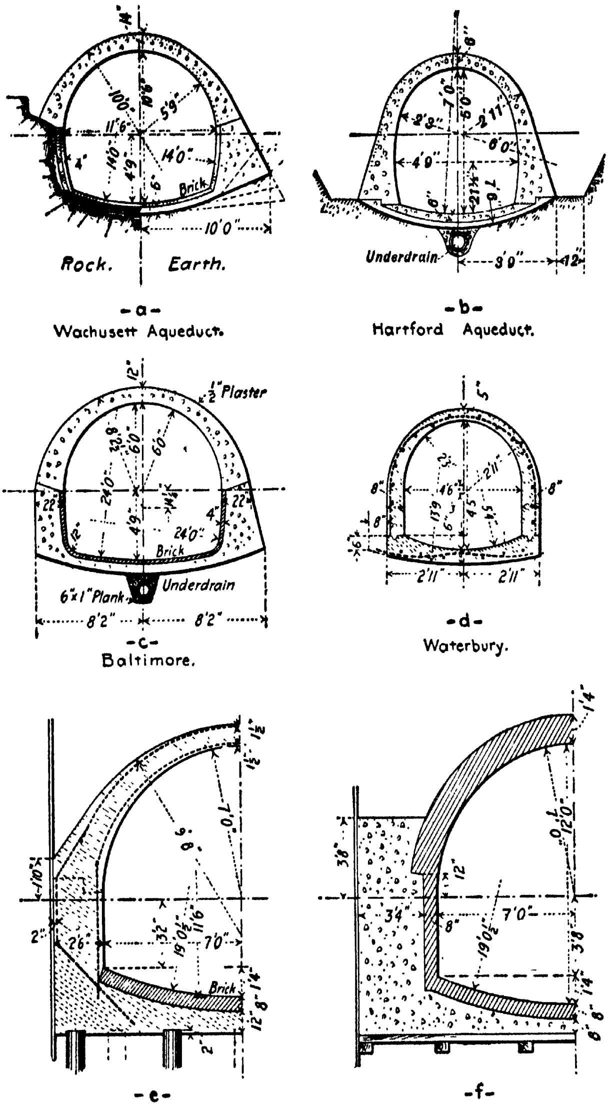


FIG. 159.—Typical basket-handle sections.



Boston

FIG. 160.—Typical horseshoe sections.

atter sewer, 9 ft. 8 in. and 11 ft. 8 in. for firm- and soft-ground sections respectively. A 5-ft 8-in. × 5-ft. 11-1/2-in. section was 8 in. thick at crown and 16 in. at springing line for firm-ground section and 28 in. for soft-ground section. Thickness of invert below vitrified shale brick lining, 6 in.; maximum width of masonry, 8 ft. 4 in. and 10 ft. 4 in., respectively for the firm- and soft-ground sections.

Fig. 159*e*.—Jersey City Water Supply Co., Jersey City, N. J., aqueduct, 1903, E. W. Harrison, Chief Eng. Basket-handle section. 8 ft. 6 in. × 8 ft. 6 in. Left half of illustration, construction in soft earth; right half, section built on embankment. Transverse steel reinforcement, 3/8-in. twisted rods 12 in. on centers; longitudinal bars, 1/4-in. twisted rods 24 in. on centers. Lower part of invert of soft earth section reinforced with 3-in. mesh No. 10 expanded metal; invert of section on embankment reinforced with 3/8-in. twisted bars. Where cover was about 15 ft., arch was 8 in. thick at crown and side walls 14 in. thick at springing line. *Eng. Record*, Jan. 16, 1904.

Fig. 159*f*.—Newark, N. J., Water Dept., Inlet conduit in reservoir, 1901; Morris R. Sherrerd, Eng. Basket-handle section, 5 × 5 ft. Reinforcing metal, 3-in. mesh No. 10 standard. Outlet from reservoir comprises two conduits similar to one shown placed side by side, wall between two 10 in. thick and space between extrados of sections filled with concrete. Maximum width of double-conduit section, 12 ft. 6 in. Both single and double conduit sections have comparatively heavy walls to provide sufficient dead weight to overcome buoyant effect of conduits when empty and reservoir full. Test section of double conduit subjected to hydrostatic pressure up to 34 lb. per square inch without signs of weakness. *Eng. Rec.*, Dec. 12, 1903.

Fig. 160*a*.—Wachusett Aqueduct, Mass. Metropolitan Water Works, 1897, F. P. Stearns, Chief Eng. Horse-shoe section. 11 ft. 6 in. × 10 ft. 6 in. The figure shows construction in rock cut, and by full and dotted lines the types in earth from hardpan to soft foundations. Cover shallow; about 4 ft. for a considerable distance. *Eng. News*, Feb. 25, 1897.

Fig. 160*b*.—Hartford, Conn., Aqueduct, 1912. C. M. Saville, Chief Eng. Horse-shoe section, 4 ft. 9 in. × 6 ft. 9-1/2 in. Largely in earth trench with about 3-ft. cover.

Fig. 160*c*.—Baltimore, Md., Outfall Sewer, 1907. Calvin W. Hendrick, Chief Eng. Horse-shoe shape, 12 ft. × 10 ft. 9 in. Left half, construction used in tunnel or sheeted trench; right half, type in loose earth or fill. *Eng. Rec.*, Feb. 8, 1908.

Fig. 160*d*.—Waterbury, Conn.; main intercepting sewer, 1907, R. A. Cairns, City Eng. Horse-shoe shape, 4 ft. 6 in. × 4 ft. 5 in. Transverse steel reinforcing bars 3/8 in. square 6 in. c. to c.; longitudinal bars, 5/16 in. square. On soft bottom footing extended 8 in. outside vertical walls. About 1500 ft. in river bed constructed with much heavier section forming retaining wall. *Eng. Record*, April 4, 1908.

Fig. 160*e*.—Boston, Mass., Tenean Creek conduit, 1909, E. S. Dorr, Chief Eng. Horse-shoe shape, 14 ft. × 11 ft. 6 in. Transverse steel 3/4-in. twisted bars 12 in. c. to c. The conduit was constructed on piles, 4 to a bent placed 5 ft. c. to c.

Fig. 160*f*.—Boston, Mass., Tenean Creek Sewer. Brick horse-shoe conduit, 14 ft. × 12 ft. This is much older than Fig. 160*e* and affords an interesting comparison between the former methods, involving the use of a brick arch with concrete backing, and the modern type of reinforced concrete construction. Structure built on timber platform of 4-in. plank on 6 × 8-in. sills.

Fig. 161*a*.—Cambridge, Mass., Marginal conduit, 1908, Charles River Basin Comm., Hiram A. Miller, Chief Eng. Horse-shoe section, 6 × 5 ft.

Fig. 161*b*.—Syracuse, N. Y., Main Intercepting Sewer, 1910, Intercepting Sewer Board, Glenn D. Holmes, Chief Eng. Horse-shoe section, 6 ft. 7 in. × 7 ft. 3 in., equivalent to 87-in. circular. Smaller sections built of same general form with thinner masonry. The 4-ft. 10-in. × 5-ft. 4-in. section had 6-in. crown and invert thickness and 10-in. side-wall thickness.

Fig. 161*c*.—Mass. Metropolitan Sewerage Comm., North Metropolitan Sewer, Section 22, 1892, Howard A. Carson, Chief Eng. Horse-shoe or basket-handle section, 3 ft. × 3 ft. 2 in. Constructed generally in very fine running sand on 3-in. plank platform on 8 × 8-in. caps, on piles 3 ft. c. to c., two piles to bent. For short distance on clay foundation sewer built on cradle of 1-in. boards laid on 2 × 4-in. ribs; constructed entirely of two rings of brick masonry. *Eng. News*, Feb. 8, 1894.

Fig. 161*d*.—Lancaster, Pa., 1903, Samuel M. Gray, Eng. Horse-shoe section, 7 ft. 6 in. × 8 ft. 4 in. The type shown in left half contains 32.6 cu. ft. brickwork and 4.8 cu. ft. con-

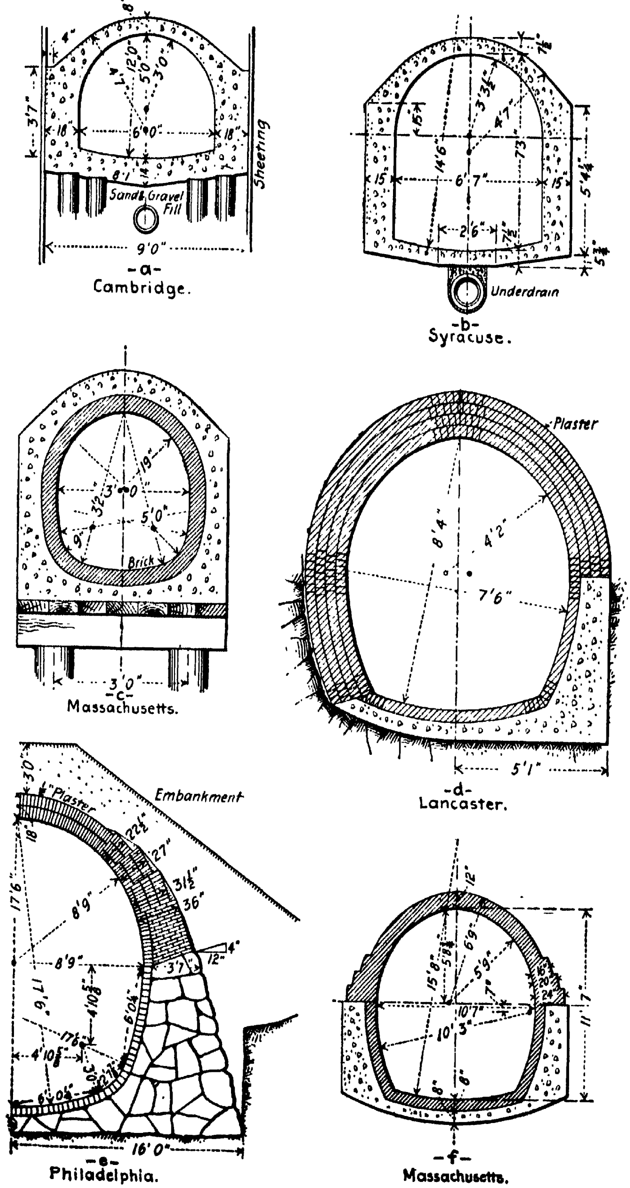


FIG. 161.—Typical horseshoe sections.

crete per linear foot; type shown in right half contains 24.7 cu. ft. brickwork and 18.5 cu. ft. concrete. Sectional area of waterway, 50.4 sq. ft. If constructed of concrete, sections could be reduced to 7 ft. 4 in. × 8 ft. 2 in. with the same general shape. Concrete section in rock, thickness was 6 in. at crown, 9 in. at springing line and 6 in. at invert below vitrified brick lining. Section reinforced with 3-in. No. 10 expanded metal. Section contained 2 cu. ft. of brickwork and area of waterway was 49.06 sq. ft.

Fig. 161*e*.—Philadelphia, Pa., Annsbury St. Sewer, 1909, Geo. S. Webster, Chief Eng., Bureau of Surveys. Horse-shoe section, 17 ft. 6 in. × 17 ft. 6 in. Built generally in shallow cut with 3-ft. cover over the top of the sewer. Fig. 155*b* shows another type of brick construction of interest in comparison with that in this figure.

Fig. 161*f*.—Mass. Metropolitan Sewerage Comm., South Metropolitan High Level Sewer, 1902, William M. Brown, Chief Eng. Horse-shoe type, 10 ft. 7 in. × 11 ft. 7 in. Concrete used generally for side walls and invert backing, with one or two rings of brick lining, depending upon amount of ground water. Concrete occasionally used for arch, but arches were mostly 12-in. brickwork.

Fig. 162*a*.—Louisville, Ky., Beargrass Intercepter, Section A., 1908, J. B. F. Breed, Chief Eng. Horse-shoe section, 6 ft. 6 in. × 6 ft. 1-1/2 in. Left half, construction in open cut with 3 to 11-ft. cover; the right half, type in tunnel. Excavation in clay and sand; water encountered in open cut The steel reinforcing bars for the open-cut section were as follows: Transverse arch bars, 1/2 in. square, 9-1/2 in. c. to c., likewise side wall and invert bars: longitudinal bars were 1/2 in. square, 13-1/4 in. c. to c. One section built on piles driven about 20 ft. in bents of three each, 4 ft. on centers. Portion of tunnel section built on concrete piles, in holes bored with augur, making the finished hole 10 in. in diameter. Material encountered a fill of clay and mud. Vertical steel reinforcement placed in each hole and hole then filled with concrete. Some material encountered was so wet and mucky, that concrete was placed through iron casing withdrawn as concrete filled hole. In another section 12-in. wrought-iron pipe casing was driven and concrete placed in it without reinforcement. Most tunnel work was in dry loose running sand. The entire cross-section of the tunnel was backfilled with concrete to a point 1 ft. above springing line of sewer arch.

Fig. 162*b*.—Louisville, Ky., 34th Street Outlet Sewer, 1909, J. B. F. Breed, Chief Eng. Horse-shoe section, 7 ft. × 6 ft. 8 in. Maximum cover, about 25 ft.; average, about 10 ft. Excavation largely in sand, gravel and clayey loam with some loose rocks. Transverse reinforcing bars, 1/2 in. round, 9 in. on centers; longitudinal bars, 5/8 in. round, spaced as shown. Interior of sewer below springing line lined with vitrified brick. Structure built for considerable distance on Simplex concrete piles.

Fig. 162*c*.—Louisville, Ky., Northwestern sewer, Section B1, Contract No. 53, 1910, J. B. F. Breed, Chief Eng. Horse-shoe shape, 13 ft. 6 in. × 9 ft., equivalent to 11-ft. 3-in. circular sewer. All transverse bars 3/4 in. square 9 in. c to c.; longitudinal bars 3/4 in. spaced as shown. Excavation mainly in sand and gravel with some yellow clay.

Fig. 162*d*.—Louisville, Ky., Northwestern Sewer, Section B2, Contract No. 54, 1910, J. B. F. Breed, Chief Eng. Horse-shoe section, 9 × 9 ft. Transverse reinforcing bars, 1/2 in. square, 12 in. on centers; longitudinal steel bars, 5/8 in. square, spaced as shown. Excavation in clay and sand. Average cover about 13 ft.

Fig. 162*e*.—Borough of the Bronx, New York City. Horse-shoe shaped, 8 ft. 6 in. × 6 ft. 9-1/2 in.; very heavy construction for soft foundation. Transverse arch bars, 3/4 in. square 10 in. c. to c.; transverse invert bars, 1-1/8 in. square, 10 in. c. to c.; longitudinal bars in arch, 1/2 in., 12 in. c. to c.; longitudinal bars in foundation over piles, 1/2 in. square, 6 in. c. to c.; transverse reinforcement in concrete caps over piles, 5/8 in. bars.

Fig. 162*f*.—New Bedford, Mass., Outfall Sewer, 1912, Wm. F. Williams, City Eng. Horse-shoe section, 7 ft. 8 in. × 7 ft. Right half, construction for 2 to 8 ft. cover; left half, heavier section for more severe loading. Reinforcement for right half; transverse bars at intrados, 1/2 in. round, 6-1/4 to 7-1/2 in. c. to c., depending on depth of fill; extrados bars 1/2 in. round, 5-1/2 to 8-3/4 in. c. to c.; interior side wall and invert bars, 5/8 in. round and 8-1/2 to 16 in. c. to c.; longitudinal bars 1/2 in. square twisted, 12 in. c. to c. Materials per linear foot of sewer; 27.3 cu. ft. concrete, 84 lb. reinforcing bars, for cover from 2 to 5 ft. and 88.65 lb. for cover from 5 to 8 ft. Reinforcing for left half; transverse intrados bars, 3/4 in. round 9-1/2 in. c. to c.; transverse extrados bars, 3/4 in. round, 12 in. c. to c.; interior side wall bars, 5/8 in. round, 8-1/2 to 16 in. c. to c.; interior invert bars, 5/8 in. round 8-1/2 to 16 in. c. to c.; exterior invert and side-wall bars, 5/8 in. round 6-3/4 in. c. to c.; longi-

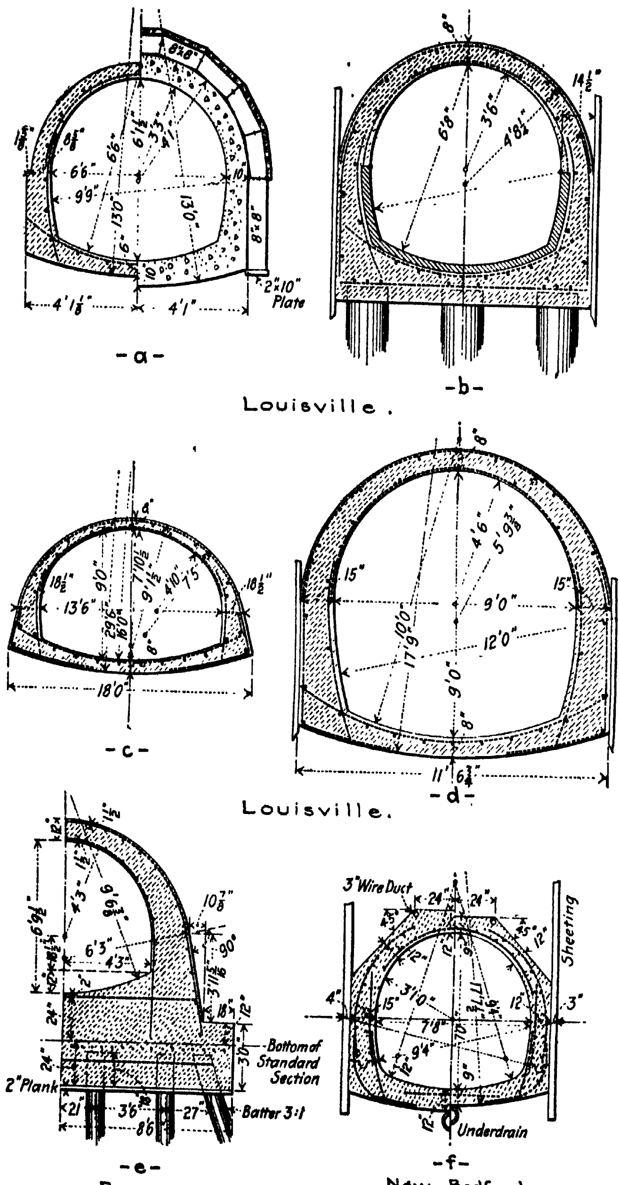


Fig. 162.—Typical horseshoe sections.

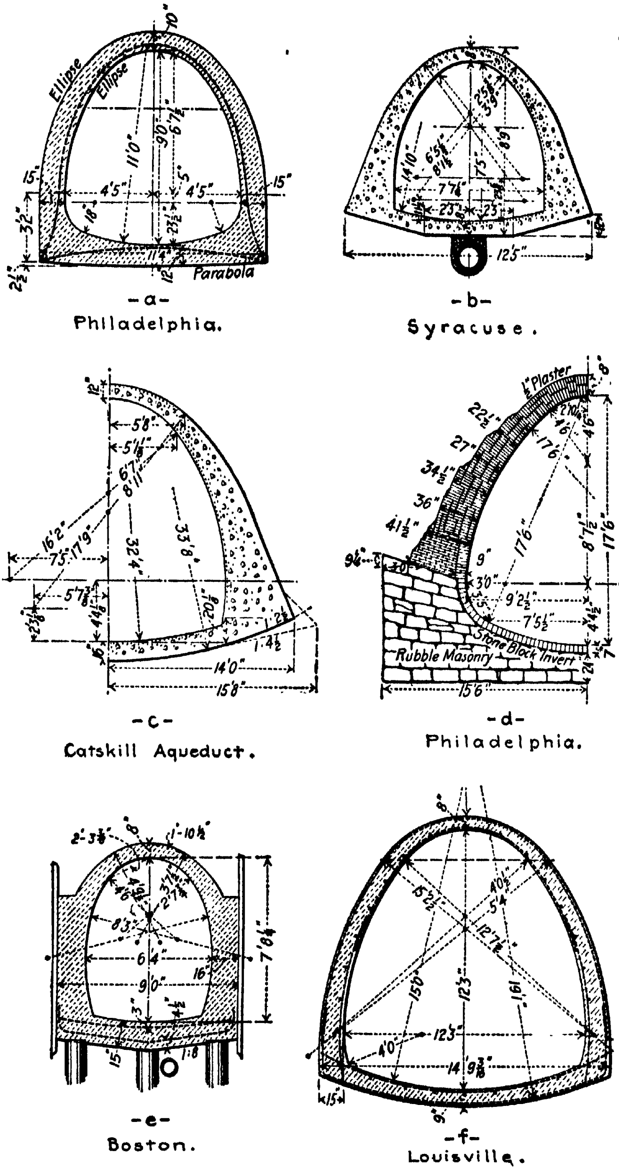


FIG. 163.—Typical semi-elliptical sections.

tudinal bars, 1/2 in. square twisted. Material per linear foot of sewer, 34.75 cu. ft. concrete; 90.6 lb. of reinforcing steel.

Fig. 163*a*.—Philadelphia, Pa., Torresdale filtered water conduit. Semi-elliptical section, 9 × 9 ft. to stand 20 ft. head of water. Reinforcement, expanded metal 6-in. mesh, 5/16 × 3/8 in. Reid, "Concrete and Reinforced-concrete Construction," p. 663.

Fig. 163*b*.—Syracuse, N. Y., Main Intercepting Sewer, 1910, Glenn D. Holmes, Chief Eng. Semi-elliptical section, 7 ft. 7-1/4 in. × 7 ft. 5 in.

Fig. 163*c*.—Catskill Aqueduct, Board of Water Supply, New York City, 1908, J. Waldo Smith, Chief Eng. Semi-elliptical type, 17 ft. 6 in. × 17 ft. Cut shows construction in earth cut; dotted lines on invert show extension of section when structure was built on embankment.

"The aqueduct in dry loose earth was designed to withstand the weight of the embankment about it, whether full or empty, and also to withstand the water pressure when full without the aid of the surrounding embankment; it was designed to withstand the pressure due to the water rising from some unusual condition above the inside top of the arch. With the regular 3-ft. embankment over the top of the arch, the cut-and-cover sections are safe to carry a 12-ton road-roller, a condition that may occasionally occur at road crossings. The section is strong enough to withstand a fill not over 14 ft. deep over the top of the arch. For fills greater than this, reinforcement of steel rods will be placed in the invert to enable it to withstand the reaction caused by the heavy load. In cases where a wet earth foundation is encountered, the aqueduct will be constructed on a timber platform arranged to allow the ground-water to drain away to sumps without washing away the freshly laid concrete. Wherever the level of the ground water adjacent to the aqueduct is higher than 9 ft. above the invert, the latter is to be made thicker, in order to withstand the upward hydrostatic pressure when the aqueduct is empty. The section in compact earth was designed to effect a lower cost per linear foot of aqueduct where the character of the earth warrants, by making the bottom width narrower, by steepening the slopes of the excavation, and laying the concrete directly against the earth sides. This section can, of course, be used only where the earth is compact enough to take the thrust of the concrete arch without yielding. The section on embankment is similar to that in loose earth, except that in order to lessen the danger of settlement the base is made wide enough to distribute the load over a larger area. Provision is also made for a foundation embankment more carefully constructed than the rest of the embankment and for a possible reinforcement of the invert in such cases. The section in rock was designed so that the rock will nowhere extend nearer than 12 in. to the inside surface of the aqueduct, thus insuring stability and water tightness. Provision was made in the designs for using excavated rock in parts of the embankment at the sides and top of the aqueduct." Report of Board, 1907.

Fig. 163*d*.—Philadelphia, Pa., Mill Creek Sewer, 1912, George S. Webster, Chief Eng. Semi-elliptical or parabolic type, 18 ft. 5 in. × 17 ft. 6 in.

Fig. 163*e*.—Boston, Mass., Charles River Basin Commission, Marginal Conduit, 1905, Hiram A. Miller, Chief Eng. Semi-elliptical section, 6 ft. 4 in. × 7 ft. 8-1/4 in. Part constructed on gravel and clay bottom and remainder on piles 2 ft. apart on centers under the side walls and 4 ft. apart under the center of the invert. Lower set of reinforcing bars over piles, 1/2 in. round 12 in. c. to c.; upper set, 3/4 in. round bars 12 in. c. to c.

Fig. 163*f*.—Louisville, Ky., Southern Outfall Sewer, Section E, Contract 14, 1909, J. B. F. Breed, Chief Eng. Semi-elliptical section, 12 ft. 3 in. Average cover about 21 ft. Material excavated, sand and gravel overlaid with considerable alluvial clay; no ground water. Transverse reinforcing, 1/2-in. square bars 12 in. c. to c.; longitudinal reinforcing, 5/8-in. square bars, 21 in. c. to c. in arch and 26 in. c. to c. in invert.

Fig. 164*a*.—Borough of the Bronx, New York City. Semi-circular concrete sewer, 11 ft. 6 in. × 7 ft. 3 in. Reinforcement; transverse arch bars, 5/8 in., 10 in. c. to c.; transverse invert bars, 7/8 in., 10 in. c. to c.; intermediate side-wall bars, 1/2 in., 10 in. c. to c.; longitudinal bars in arch, 1/2 in., 12 in. c. to c.; longitudinal bars in concrete capping over piles, 3 over each outside pile, 3/4 in., 6 in. c. to c.; remaining longitudinal bars over piles, 5/8 in., 12 in. c. to c.; transverse bars or hoops around heads of piles in each bent 5/8 in. Piles driven in bents of five vertical and two brace piles, one on either side; bents spaced 3 ft. 6 in. to 4 ft.; piles in bent spaced 3 ft. 3 in. c. to c. *Trans. Am. Soc. C. E.*, vol. lxxvi, 1913, plate lxiv.

Fig. 164*b*.—Wilmington, Delaware., Clement's Run Sewer, 1903, T. Chalkley Hatton, Consulting Eng. Semi-circular sewer 10 ft. × 5 ft. 6 in., reinforced with woven wire mesh

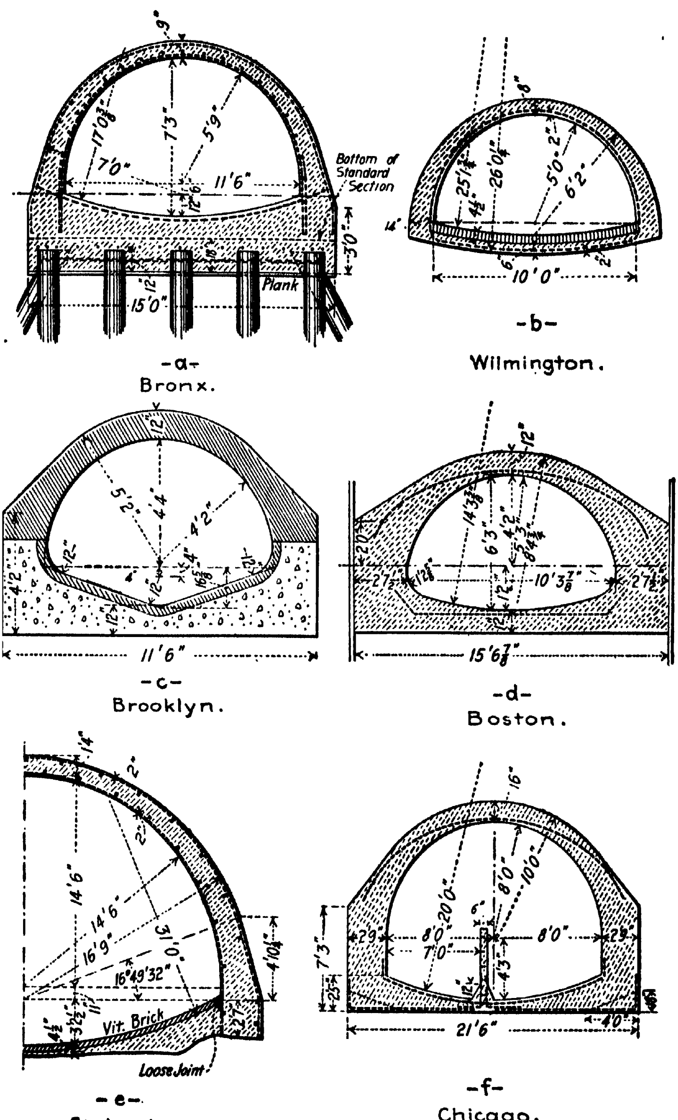


FIG. 164.—Typical semi-circular sections.

and No. 6 expanded metal. Invert lined with one course of brick. *Municipal Engineering*, October, 1904.

Fig 164*c*.—Borough of Brooklyn, New York City, 1913, E. J. Fort, Chief Eng., Semi-circular section, 8 ft. 4 in. × 5 ft. 8 in., equivalent to 78-in. circular section. Table 140 gives a comparison of the hydraulic properties of the semi-circular section and of a 78-in. circular section, both at the maximum capacity of the section.

TABLE 140.—COMPARISON OF SEMI-CIRCULAR AND CIRCULAR SECTIONS

Section	Area, sq. ft.	Wetted perimeter, ft.	Hydraulic radius, ft.	Discharge $s = 0.001$ cu. ft., sec.
Semi-circular........	32 94	18.20	1.81	151.31
Circular.............	32.35	17.20	1.93	156.25

Fig. 164*d*.—Boston, Mass., Kemp St. overflow, 1912, E S. Dorr, Chief Eng. Semi-circular section, 10 ft. 3-7/8 in. × 6 ft. 3 in. Transverse steel reinforcement, 3/4-in. bars 8 in. c. to c.

Fig. 164*e*.—St. Louis, Mo., Harlem Creek Sewer, 1908, H. F. Fardwell, Sewer Commissioner. Semi-circular section, 29 ft. × 18 ft. 7-1/2 in., to carry 15-ft. fill over arch and the heaviest railroad loading combined with 7-ft. fill. Stresses in various sections determined from analysis of circular ribs with fixed ends given in Prof. Charles E. Greene's "Trusses and Arches." Reinforcement, Johnson corrugated bars, 7/8 in. for transverse and 1/2 in. for longitudinal reinforcement. Intrados transverse bars spaced 10 in. c. to c. There were also intermediate 1/2-in. transverse bars in side walls and haunch of arch running to point 9 ft. 9-1/2 in. from top of side wall, alternately with 7/8-in. arch bars, making spacing of steel in side wall and haunch of arch, 5 in. c. to c. Extrados bars, 10 in. c. to c. Arch carried through to rock and loose joint left between side wall and invert. In earth, section considerably widened at base, invert much thicker and reinforced to distribute thrust of arch over greater area. *Eng. Record*, Dec. 14, 1907.

Fig. 164*f*.—Chicago, Ill., Sanitary District, South 52d Ave. sewer, 1913, Geo. M. Wisner, Chief Eng. Horse-shoe section, 16 ft. × 12 ft. 3 in. Dividing wall is to provide high velocities and avoid deposits by keeping dry-weather flow on one side of the wall. Stop planks at head of section divert flow to either side of dividing wall. Owing to soft ground, invert reinforced throughout entire length; upper transverse bars 1/2 in. round, 6 in. c. to c.; lower transverse bars 3/4 in. round, 12 in. c. to c. Under railroad, arch reinforced with 3/4-in. rods 6 in. on centers. Dividing wall reinforced with two rows of 1/2-in. vertical bars 6 in. on centers and 10 rows of 1/2-in. longitudinal bars 12 in. on centers. Joint between dividing wall and invert strengthened by two sets of bars bent at right angles. Height of wall above invert, 4 ft. 11 in.; wall slightly off center. *Engineering and Contracting*, Feb. 11, 1914.

Fig. 165*a*.—St. Louis, Mo., South Harlem Joint District Sewer, 1909. Horse-shoe section, 12 ft. × 9 ft. 7-1/4 in. Left half, section in rock cut; right half, section for earth. Arch designed for 20 ft. cover. Reinforcement; 5/8-in. transverse arch bars, 12 in. c. to c.; 5/8-in. intermediate side-wall bars near interior, running to point 2 ft. above springing line of arch, between arch bars; 5/8-in. transverse invert bars in earth section, 12 in. c. to c.; 1/2-in. longitudinal bars, spaced as shown. In earth materials per linear foot were: concrete, 2.62 cu. yd.; vitrified brick, 0.174 cu. yd.; transverse reinforcement, 90.2 lin. ft. 5/8-in. bars; longitudinal reinforcement, 22 lin. ft. 1/2-in. bars. For section in rock, materials were: concrete, 1.92 cu. yd.; vitrified brick, 0.174 cu. yd.; transverse steel, 73.7 lin. ft. 5/8-in. bars; longitudinal steel, 19.0 lin. ft. 1/2-in. bars.

Fig. 165*b*.—St. Louis, Mo., Dale Ave. Sewer, 1910. Rectangular section, 6 ft. 3 in. × 8 ft. Types were designed to meet three conditions. In first, natural rock surface was at or above skewback of flat arch, which had to carry whole load directly to rock. The 9-in. concrete walls were merely to smooth up sides of cut. In second case, rock was slightly below skewback and 18-in. concrete walls used. In third case, rock was more than 3 ft. below springing line; see right half of figure. The 18-in. walls, reinforced by 1-in. steel bars 12 in. on centers, were designed as beams to carry arch thrust at upper end and earth pressure below. As sewer was largely in rock, the narrow, high rectangular section was selected as most economical. Owing to depth at which sewer was built, saving in excavation due to decreased width much more than offset increased depth. *Engineering News*, Sept. 5, 1912.

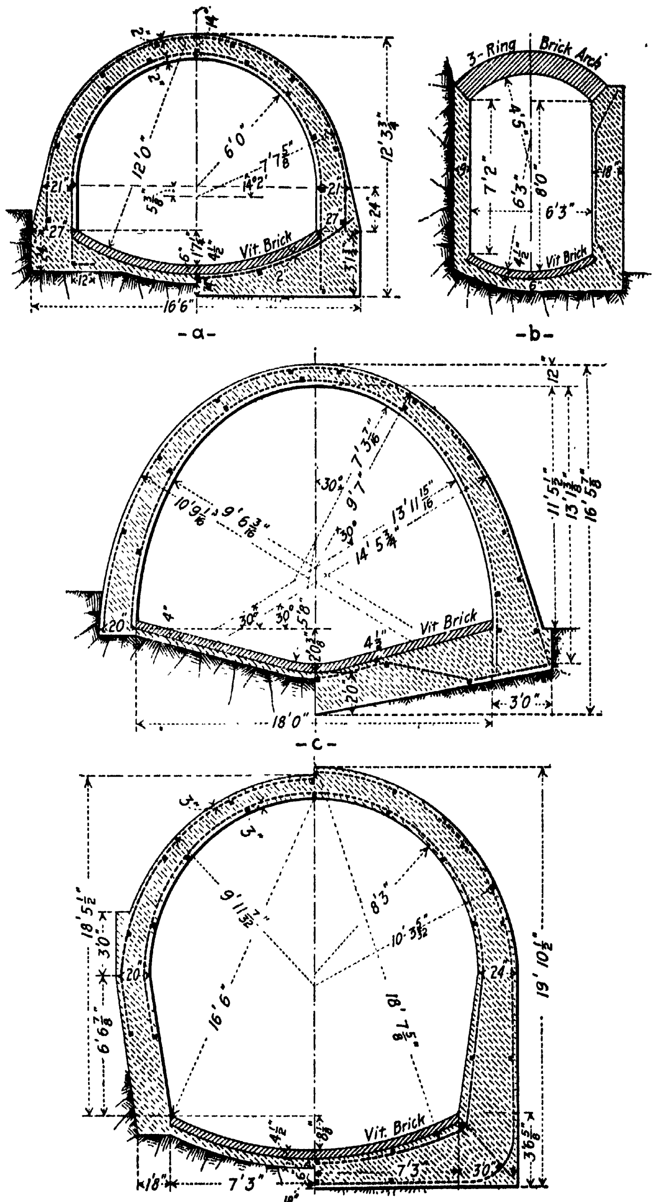


Fig. 165.—Typical St. Louis sections

Fig. 165*c*.—St. Louis, Mo., Baden Public Sewer, First Section, 1910. Five-centered, or semi-elliptical arch section, 18 ft. × 13 ft. 1-3/8 in. Left half, construction in rock cut; right half, type for earth cut. Transverse extrados reinforcing bars, 3/4 in. square, 10 in. c. to c., transverse intrados bars, ½ sq. in., 20 in. c. to c. W. W. Horner states that use of this type instead of that shown in Fig. 165*a*, has been a matter of judgment in each particular case. The five-centered arch has been preferred where loading was principally uniform earth load.

Fig. 165*d*.—St. Louis, Mo. Horse-shoe section, 16 ft. 6 in. × 16 ft. 6 in. Left half, construction where rock was encountered above springing line: right half, construction in earth cut. Arch designed to carry 25 ft. fill above crown. Reinforcement; transverse intrados and extrados bars for right half, 3/4 in. square, 10 in. on centers; for left half transverse extrados bars 5/8 in. square, 10 in. on centers; transverse invert bars, 3/4 in. square, 5 in. on centers; upper transverse invert bars extend to point 2 ft. below springing line of arch. Longitudinal bars 3/4 in. square, spaced as shown, except two bars at side angle of invert and side walls, which were 1-1/4 in. round.

Fig. 166*a*.—Harrisburg, Pa., Paxton Creek Intercepting Sewer, 1903, James H. Fuertes, Consulting Eng. Parabolic section, 6 × 5 ft.; also smaller section of same type, 5 ft. 1-1/2 in. × 3 ft. 9 in., with same thickness of masonry. Sewer crosses swamp and meadow land

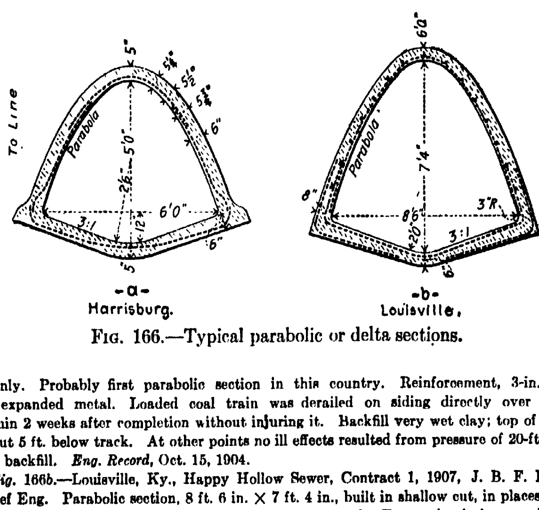


FIG. 166.—Typical parabolic or delta sections.

mainly. Probably first parabolic section in this country. Reinforcement, 3-in. No. 10 expanded metal. Loaded coal train was derailed on siding directly over sewer within 2 weeks after completion without injuring it. Backfill very wet clay; top of sewer about 5 ft. below track. At other points no ill effects resulted from pressure of 20-ft. very wet backfill. *Eng. Record*, Oct. 15, 1904.

Fig. 166*b*.—Louisville, Ky., Happy Hollow Sewer, Contract 1, 1907, J. B. F. Breed, Chief Eng. Parabolic section, 8 ft. 6 in. × 7 ft. 4 in., built in shallow cut, in places more than half the sewer being above natural surface of ground. Excavation in loam and clay. Section considered especially advantageous for conditions, on account of economy of space in embankment section, and strong arch afforded. Transverse reinforcement, 1/2-in. bars, 12 in. on centers; longitudinal reinforcement, 1/2-in bars, spaced as shown, approximately 12 in. on centers. Sewer may be covered by fill of 30 ft. hereafter.

Fig. 167*a*.—Beargrass Creek Drain, Section A, Contract 36, 1909, J. B. F. Breed, Chief Eng. Rectangular section, 6 ft. × 4 ft. 9 in., constructed in alluvial clay requiring foundation of oak piles driven in bents of three, spaced 3 ft. 2 in. c. to c. Transverse reinforcement in flat roof, 1/2 in. square bars, 6 in. on centers; remainder of transverse steel in side walls and invert, ½-in. square bars, 9 in. on centers; longitudinal reinforcement, ½-in. square bars spaced approximately 13 in. on centers: longitudinal reinforcement over each pile, 1/2 in. square bars.

Fig. 167*b*.—Harrisburg, Pa., Susquehanna River intercepting sewer, 1912, James H. Fuertes, Consulting Eng. Rectangular section, 3 ft. 6 in. × 3 ft. 6 in., reinforced transversely

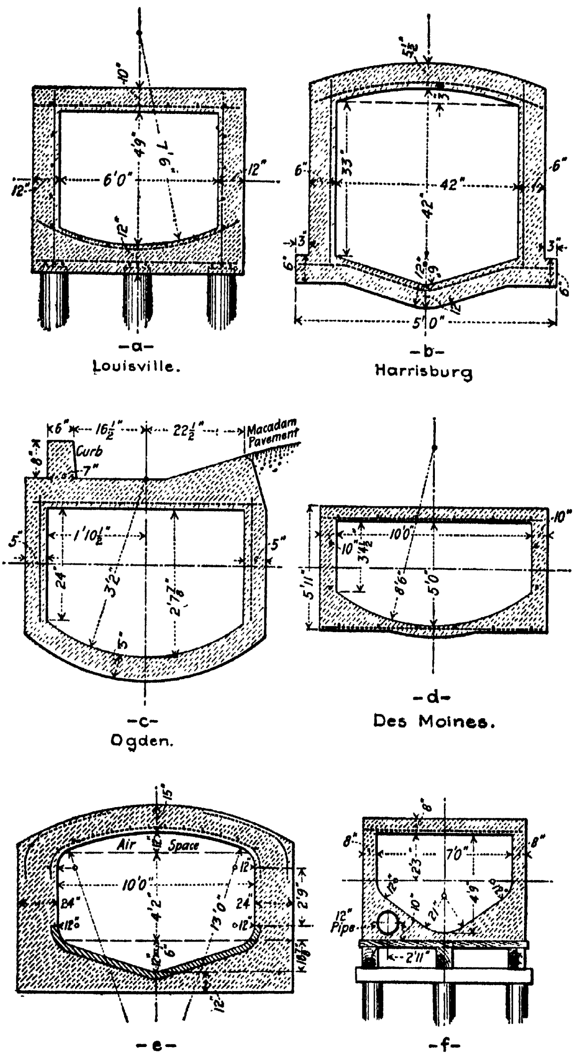


FIG. 167.—Typical rectangular sections.

with 1/2 in. square bars, 7 in. on centers, except where sewer was on rock bottom, when reinforcement in invert was omitted. Rectangular section chosen on account of proximity of sewer to surface of ground. *Eng. Record*, Feb. 24, 1912.

Fig. 167*c*.—Ogden, Utah, 1907, A. F. Parker, City Eng. Rectangular conduit, 3 ft. 9 in. × 2 ft. 7-7/8 in., with top practically at surface of ground in street and located so that gutter and curb forms part of conduit. Under street crossings side walls reduced in height to 16-1/8 in. and the top curved in form of arch like invert. Roof reinforced with ½-in. rods, 8 in. on centers; side walls reinforced with 3/8-in. rods 16 in. on centers. *Eng. Record*, Jan. 18, 1908.

Fig. 167*d*.—Des Moines Ia., Ingersoll Run Sewer, 1905, John W. Budd, City Eng. Rectangular section, 10 ft. × 5 ft, is selected on account of proximity of grade line of sewer to street surface. Transverse steel in flat roof, 1/2-in. corrugated bars 24 in. on centers; in invert, 1/2-in. corrugated bars 12 in. on centers. Longitudinal bars, 1/2-in. square, spaced as shown. *Eng. Record*, April 28, 1906.

Fig. 167*e*.—Borough of Brooklyn, New York City, 1913, E. J. Fort, Chief Eng. Rectangular section 10 ft. × 6 ft. 8 in., approximately equivalent to 102-in. circular sewer. Table 141 compares the hydraulic properties of the rectangular section filled to within 12 in. of the crown, with the properties of a 102-in. circular sewer.

TABLE 141.—COMPARISON OF HYDRAULIC PROPERTIES OF RECTANGULAR AND CIRCULAR SECTIONS.

Section	Wetted area, sq. ft.	Wetted perimeter, feet	Hydraulic radius, feet	Discharge in cu. ft., sec., $s = 0.001$
10 ft. × 6 ft. 8 in. rectangular.	49.12	18.55	2.65	294.60
102 in. circular......	55.43	22.51	2.41	319.64

Fig. 167*f*.—Hoboken, N. J., 1913, James H. Fuertes, Consulting Eng. Rectangular section, 7 ft. × 4 ft. 9 in., is of particular interest on account of V-shaped waterway provided for low flows. Rectangular section with flat top selected on account of lack of head room between surface of ground and top of sewer. Sewer in soft foundation has timber platform of 4-in. planks on 10 × 12-in. stringers on 3 × 8-in. caps, two to each pile bent. Piles spaced 3 ft. 8 in. c. to c., 3 piles to a bent. Roof reinforced with 5/8-in. bars 10 in. c. to c.

Fig. 168*a*.—Lancaster, Pa., 1903, Samuel M. Gray, Eng. Semi-circular, 12 × 6-ft., section with 24-in. half-round dry-weather flow channel or "cunette." Two types designed, one with concrete arch reinforced with 3-in. mesh No. 10 expanded metal, and the other with an arch of four rings of brickwork.

Fig. 168*b*.—Louisville, Ky., Southern Outfall Sewer, Section A, Contract 6, 1908, J. B. F. Breed, Chief Eng. Horse-shoe section, 8 ft. wide with 3-ft. half-round cunette. Transverse intrados arch bars and side wall bars, 5/8-in. square, 12 in. on centers; extrados arch bars and exterior side wall bars, 1/2-in., 12 in. on centers; upper and lower transverse invert bars, ½-in. square, 12 in. on centers: longitudinal bars, 5/8-in. square; spaced as shown. Section on incline of about 30 deg. to horizontal, and cunette used to confine dry-weather flow on account of high velocity. Invert of cunette lined with 36-in. vitrified clay channel pipe.

Fig. 168*c*.—Washington, D. C., New Jersey Ave., Trunk Sewer, 1902, designed in office of Engineer Commissioner of District of Columbia. Semi-circular, 18 × 16-ft. section with 9-ft. half-round cunette.

Fig. 168*d*.—Brussels, Belgium, Maelbeek Creek Storm-water Sewer, 1895. Horse-shoe shape, 14 ft. 9-1/2-in. × 12 ft. 1-3/4-in., with 6 ft. 8 in. wide cunette. Interior of sewer lined with 1/4-in. cement plaster and exterior covered with 3/4-in. coating of cement mortar. *Eng. News*, Mar. 26, 1896.

Fig. 168*e*.—Syracuse, N. Y., Harbor Brook Intercepting Sewer, 1912, Glenn D. Holmes, Chief Eng. U-shaped 30-in. section. Left half, construction in firm material, right-half section on pile foundation. Flat slab top built separately and set in place, joints being filled with mortar. Slab 12 in. wide reinforced with 3/8-in. square bars 6 in. on centers. A sewer of practically same design 3 ft. wide at top was constructed in Lynn, Mass., in

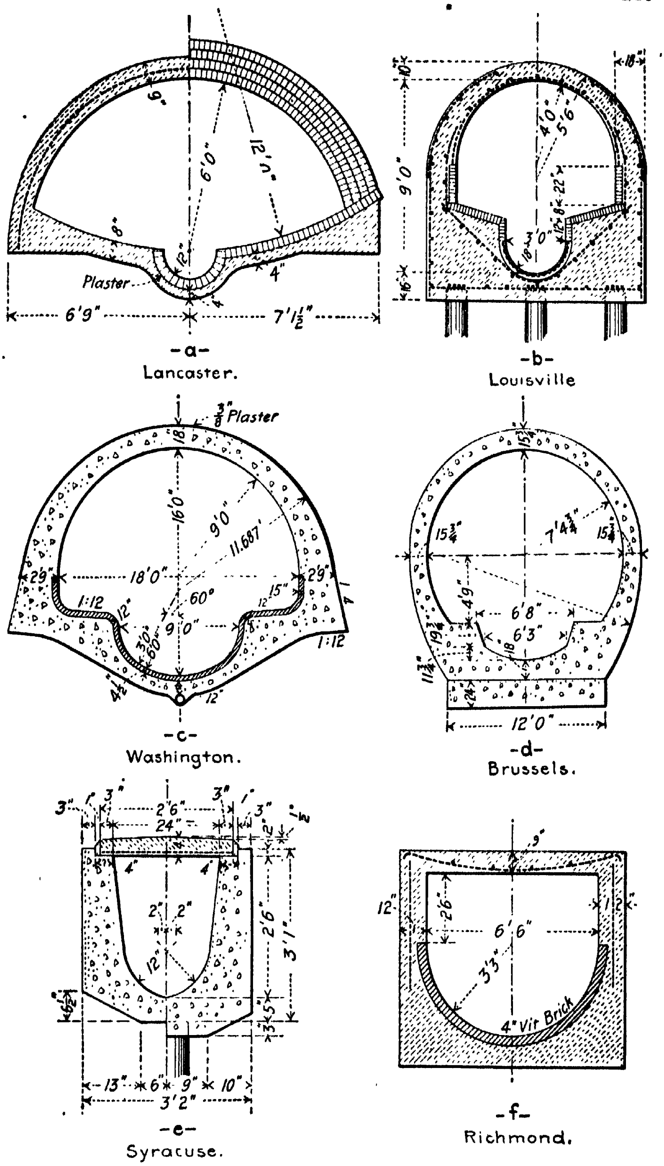


FIG. 168.—Cunette and U-shaped sewer sections.

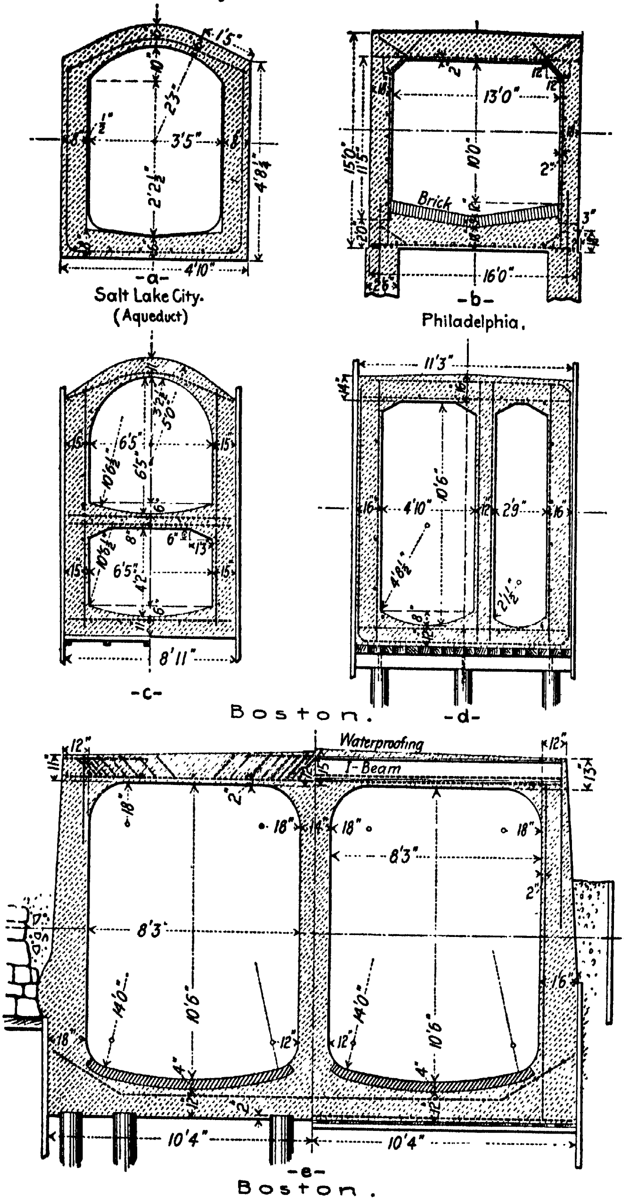


FIG. 169.—Typical rectangular sections.

1909, from plans of C. H. Dodd, Chief Draftsman, Boston Sewer Department, who also designed similar section 2 ft. wide on top for Boston in 1908.

Fig. 168*f*.—Borough of Richmond, New York City, District 6A Trunk Sewer, 1907, Louis L. Tribus, Comr. of Public Works. U-shaped 6 ft. 6 in. semi-circular section. Side walls reinforced with No. 10 expanded metal and flat slab roof reinforced with 3/4-in. old style Johnson bars 6 in. on centers transversely and 18 in. longitudinally. General surface of land below top of sewer. *Eng. Record*, Nov. 2, 1907.

Fig. 169*a*.—Salt Lake City, Big Cottonwood water conduit, 1907, L. C. Kelsey, City Eng. Rectangular section, 3 ft. 5 in. × 4 ft. 5-1/2-in. Figure shows construction in fill; similar section used in excavation, except reinforcing bars were placed nearer interior. In tunnel, section resembled that shown but lacked reinforcement. *Engineering and Contracting*, Aug. 5, 1908.

Fig. 169*b*.—Philadelphia, Pa., Devereaux St. Sewer, 1909, Geo. S. Webster, Chief Eng. Section constructed in mud through low land on 2-1/2 × 5-ft. piers spaced 15 ft. c. to c. longitudinally and 11 ft. 6 in. apart transversely. Sewer protected by embankment with 3 ft. cover. Transverse reinforcement of flat slab top, 1 in. square bars 6 in. c. to c.; both ends of every other rod bent up at an angle of 30 deg. 2 ft. 9 in. from either end; side-wall bars, 5/8-in., square, 6 in. c. to c.; transverse invert bars, 7/8-in. square, 6 in. c. to c. Longitudinal bars in roof and side walls, 5/8-in. square, approximately 18 in. c. to c.: longitudinal bars in invert, 5/8-in. square, except at either end over piers they were 1 in. square, 6 in. c. to c. Between invert and pier were three vertical dowels 1 in. square, 12 in. c. to c. Roof pitched 2 in. from center to outside, plastered with 1-in. cement mortar.

Fig. 169*c*.—Boston, Mass., South End Sewer Improvement, Section 2, Union Park St., 1913, E. S. Dorr, Chief Eng. Double conduit rectangular sections, 6 ft. 5 in. × 6 ft. 5 in., and 6 ft 5 in. × 4 ft. 2 in. Double structure required by limited space for construction of conduits. Transverse bars, 7/8-in., 12 in. c. to c.; longitudinal bars, 1/2-in., spaced as shown. Section constructed on platform of 2-in. plank laid on 3 × 4-in. sills.

Fig. 169*d*.—Boston, Mass., South End Sewer Improvement, Section 4. Albany St.; 1913; E. S. Dorr, Chief Eng. Gravity sewer, 4 ft. 10 in. × 10 ft. 6 in.; force main 2 ft. 9 in. × 10 ft. 6 in.; rectangular sections, double conduit. Section constructed on 4-in. planks on 8 × 8-in. caps on three-pile bents. Transverse reinforcement, 7/8-in. rods, spaced 24 in. on centers in upper roof, lower invert and vertically in division wall; other places, 12 in. on centers. Longitudinal reinforcement, 1/2-in. rods spaced as shown. Section shows limits to which it is sometimes necessary to go where space is very much restricted.

Fig. 169*e*.—Boston, Mass., Stony Brook Channel, 1908, E. S. Dorr, Chief Eng. Double section, 8 ft. 3 in. × 10 ft 6 in., constructed to replace old stone masonry channel, and on that account work involved special difficulties. Section with I-beams in roof used that backfilling might be placed more quickly than on section reinforced with bars. Left half, reinforcement in roof, 1/2-in. bars forming a truss unit spaced 5 in. c. to c. Right half, 10-in. I-beams in roof spaced 4 ft. on centers, with 2-in. Kahn rib metal stretched between these I-beams and lower flanges of I-beams wrapped with Kahn rib lathing. Side wall bars, 3/4-in., spaced 12 in. c. to c., invert bars, 3/4-in., spaced 8 in. c. to c. This type laid on platform of 1-in. boards on 2 × 3-in. sills.

Fig. 170*a*.—Borough of the Bronx, New York City, Broadway Outfall Sewer, 902, J. A. Briggs, Chief Eng., C. H. Graham, Engineer of Sewers. Twin semi-circular section, 15 ft. × 9 ft. 2-1/8-in., constructed largely above ground, twin section being adopted as requiring less vertical space than single large circular sewer. Depth of cover to surface of street 4 ft. Sewer constructed on concrete, timber, rubble or pile foundations, depending upon character of soil. *Eng. Record*, Nov. 11, 1905.

Fig. 170*b*.—Borough of the Bronx, New York City. Rectangular twin section 10 ft. 6 in × 8 ft. 10 in. Reinforcement; transverse roof bars, 1 in., 6 in. centers; vertical bars in outside walls, 1-1/8-in., 8 in. c. to c.; vertical bars in division wall, 1/2-in., 7-1/2 in. c. to c.; transverse invert bars, 5/8-in., 6 in. c. to c.; longitudinal bars in roof and walls, 1/2-in., spaced as shown; longitudinal bars in concrete over piles, 5/8-in., 6 in. c. to c.; tranverse bars around piles, 5/8-in. Piles spaced 3 ft. 3 in. c. to c.; 8 vertical piles to bent with two brace piles one on either side. Trans Am. Soc. C.E., vol. lxxvi, 1913, plate lxiv.

Fig. 170*c*.—Borough of Brooklyn, New York City, 1913, E. J. Fort, Chief Eng. Twin rectangular section, 11 ft. 2 in. × 8 ft. 5/8 in., approximately equivalent to 13 ft. circular sewer. Flowing completely full, rectangular section estimated to discharge 908.80 cu. ft.

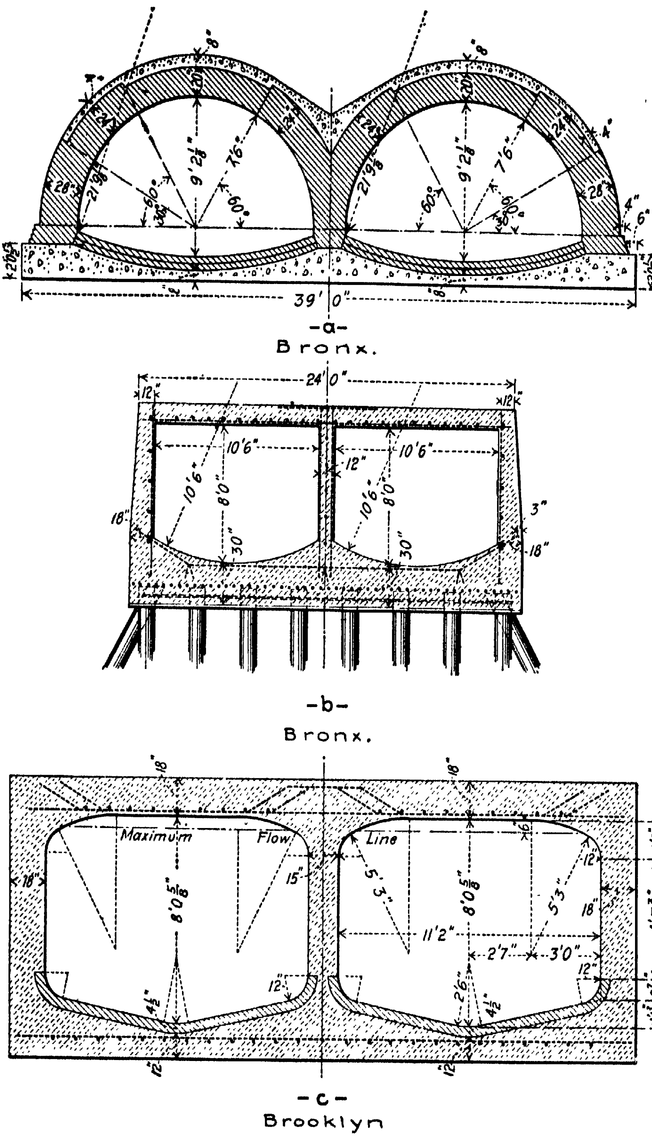


Fig. 170.—Typical double sections.

1909, from plans of C. H. Dodd, Chief Draftsman, Boston Sewer Department, who also designed similar section 2 ft. wide on top for Boston in 1908.

Fig. 168*f*.—Borough of Richmond, New York City, District 6A Trunk Sewer, 1907, Louis L. Tribus, Comr. of Public Works. U-shaped 6 ft. 6 in. semi-circular section. Side walls reinforced with No. 10 expanded metal and flat slab roof reinforced with 3/4-in. old style Johnson bars 6 in. on centers transversely and 18 in. longitudinally. General surface of land below top of sewer. *Eng. Record*, Nov. 2, 1907.

Fig. 169*a*.—Salt Lake City, Big Cottonwood water conduit, 1907, L. C. Kelsey, City Eng. Rectangular section, 3 ft. 5 in. × 4 ft. 5-1/2-in. Figure shows construction in fill; similar section used in excavation, except reinforcing bars were placed nearer interior. In tunnel, section resembled that shown but lacked reinforcement. *Engineering and Contracting*, Aug. 5, 1908.

Fig. 169*b*.—Philadelphia, Pa., Devereaux St. Sewer, 1909, Geo. S. Webster, Chief Eng. Section constructed in mud through low land on 2-1/2 ×5-ft. piers spaced 15 ft. c. to c. longitudinally and 11 ft. 6 in. apart transversely. Sewer protected by embankment with 3 ft. cover. Transverse reinforcement of flat slab top, 1 in. square bars 6 in. c. to c.; both ends of every other rod bent up at an angle of 30 deg. 2 ft. 9 in. from either end; side-wall bars, 5/8-in., square, 6 in. c. to c.; transverse invert bars, 7/8-in. square, 6 in. c. to c. Longitudinal bars in roof and side walls, 5/8-in. square, approximately 18 in. c. to c.: longitudinal bars in invert, 5/8-in. square, except at either end over piers they were 1 in. square, 6 in. c. to c. Between invert and pier were three vertical dowels 1 in. square, 12 in. c. to c. Roof pitched 2 in. from center to outside, plastered with 1-in. cement mortar.

Fig. 169*c*.—Boston, Mass., South End Sewer Improvement, Section 2, Union Park St., 1913, E. S. Dorr, Chief Eng. Double conduit rectangular sections, 6 ft. 5 in. × 6 ft. 5 in., and 6 ft 5 in. × 4 ft. 2 in. Double structure required by limited space for construction of conduits. Transverse bars, 7/8-in., 12 in. c. to c.; longitudinal bars, 1/2-in., spaced as shown. Section constructed on platform of 2-in. plank laid on 3 × 4-in. sills.

Fig. 169*d*.—Boston, Mass., South End Sewer Improvement, Section 4. Albany St.; 1913; E. S. Dorr, Chief Eng. Gravity sewer, 4 ft. 10 in. × 10 ft. 6 in.; force main 2 ft. 9 in. × 10 ft. 6 in.; rectangular sections, double conduit. Section constructed on 4-in. planks on 8 × 8-in. caps on three-pile bents. Transverse reinforcement, 7/8-in. rods, spaced 24 in. on centers in upper roof, lower invert and vertically in division wall; other places, 12 in. on centers. Longitudinal reinforcement, 1/2-in. rods spaced as shown. Section shows limits to which it is sometimes necessary to go where space is very much restricted.

Fig. 169*e*.—Boston, Mass., Stony Brook Channel, 1908, E. S. Dorr, Chief Eng. Double section, 8 ft. 3 in. × 10 ft 6 in., constructed to replace old stone masonry channel, and on that account work involved special difficulties. Section with I-beams in roof used that backfilling might be placed more quickly than on section reinforced with bars. Left half, reinforcement in roof, 1/2-in. bars forming a truss unit spaced 5 in. c. to c. Right half, 10-in. I-beams in roof spaced 4 ft. on centers, with 2-in. Kahn rib metal stretched between these I-beams and lower flanges of I-beams wrapped with Kahn rib lathing. Side wall bars, 3/4-in., spaced 12 in. c. to c., invert bars, 3/4-in., spaced 8 in. c. to c. This type laid on platform of 1-in. boards on 2 × 3-in. sills.

Fig. 170*a*.—Borough of the Bronx, New York City, Broadway Outfall Sewer, 902, J. A. Briggs, Chief Eng., C. H. Graham, Engineer of Sewers. Twin semi-circular section, 15 ft. × 9 ft. 2-1/8-in., constructed largely above ground, twin section being adopted as requiring less vertical space than single large circular sewer. Depth of cover to surface of street 4 ft. Sewer constructed on concrete, timber, rubble or pile foundations, depending upon character of soil. *Eng. Record*, Nov. 11, 1905.

Fig. 170*b*.—Borough of the Bronx, New York City. Rectangular twin section 10 ft. 6 in × 8 ft. 10 in. Reinforcement; transverse roof bars, 1 in., 6 in. centers; vertical bars in outside walls, 1-1/8-in., 8 in. c. to c.; vertical bars in division wall, 1/2-in., 7-1/2 in. c. to c.; transverse invert bars, 5/8-in., 6 in. c. to c.; longitudinal bars in roof and walls, 1/2-in., spaced as shown; longitudinal bars in concrete over piles, 5/8-in., 6 in. c. to c.; tranverse bars around piles, 5/8-in. Piles spaced 3 ft. 3 in. c. to c.; 8 vertical piles to bent with two brace piles one on either side. Trans Am. Soc. C.E., vol. lxxvi, 1913, plate lxiv.

Fig. 170*c*.—Borough of Brooklyn, New York City, 1913, E. J. Fort, Chief Eng. Twin rectangular section, 11 ft. 2 in. × 8 ft. 5/8 in., approximately equivalent to 13 ft. circular sewer. Flowing completely full, rectangular section estimated to discharge 908.80 cu. ft.

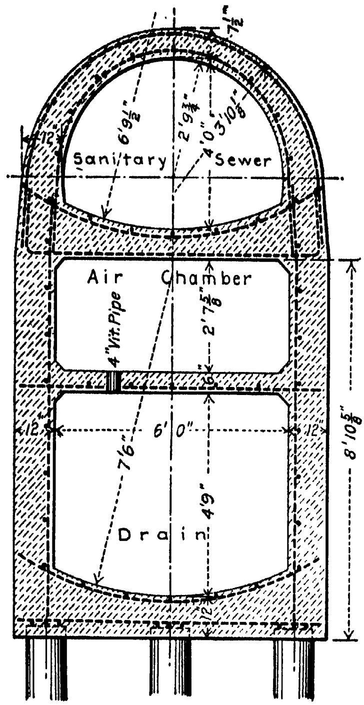


FIG. 172.—Compound sewer section, Louisville.

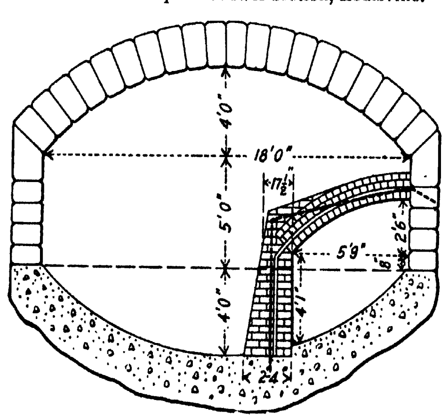


FIG. 173.—Millbrook intercepter, Worcester.

piles for both structures. Piles spaced 3 ft. 2 in. c. to c. transversely, three piles to bent. Fall of drain and sewer toward opposite ends; sewer and drain were separated at upper end by very low chamber, which gradually increased, due to increasing difference in elevations of sewer and drain. Material excavated almost wholly alluvial clay. Bents 3 ft. c. to c. Structure built in four operations, invert of drain first, after which the side walls and top of the drain were constructed. Following the completion of the drain the side walls of the air chamber between the sewer and the drain and the invert of the sewer were built as a third operation, after the completion of which the concrete was placed in the arch of the sewer. The transverse reinforcement in the side walls was 3/4-in. bars spaced 9 in. on centers. The remainder of the transverse reinforcement consisted of half-inch bars 9 in. on centers. The longitudinal bars were half inch, spaced as shown. Over each pile bent there were three half-inch transverse bars, spaced 4 in. c. to c.

Fig. 173.—Worcester, Mass., Millbrook Intercepting Sewer, 1897, Frederick A. McClure, City Engineer. Larger section is old trunk sewer constructed in 1880 of quarried stone with concrete invert, laid through ledge and occupying so much of street that it was deemed impracticable to parallel it with intercepters. Accordingly conduit was designed to accommodate flow of sewage inside large sewer. The brick section was constructed inside cofferdam; average depth of flow in main sewer during construction about 3 ft. See annual report of Superintendent of Sewers, Worcester, 1899.

SELECTION OF MATERIALS OF CONSTRUCTION

Materials for Arches.—In the older sewerage systems will be found examples of large sewer arches constructed of stone blocks. An example is Fig. 173, a section of the Millbrook conduit in Worcester, Mass. One reason for choosing stone blocks was their availability and lower cost as compared with brickwork for large arches, and further, in the days when such sewers were constructed, concrete and reinforced concrete were used little, if at all. Even more recently, rubble masonry has been used to a considerable extent, especially in Philadelphia, on account of its relative economy. Its use has, however, been largely for foundations and masonry below the springing line. Stone blocks have now been practically superseded by other materials for sewer arches. Although stone arches have fewer joints it is more difficult to obtain tight joints, and consequently the leakage is apt to be larger than when other materials are used.

Brick masonry is still used to a great extent for sewer arches, principally on account of its economy in certain cases and the ease with which brick masonry can be handled in tunnels and restricted places. Brick arches, owing to their greater number of joints, are more liable to settlement than concrete and unless special means are employed in bonding the brick the strength of the structure may be more uncertain.

In the construction of brick arches, three general types of bonding have been used. In the first, the arch is built of concentric rings of brick with all bricks laid as stretchers; this is sometimes called "rowlock" bond. In the second type the brick are laid part as stretchers and part as headers, as in ordinary brick-wall construction, with radial joints in which the outer end of the joint is thickened by increasing the thickness of the mortar or by insertion of thin pieces of slate. In the

third method the masonry is divided into blocks or sections, Figs. [illegible], 161*e* and 163*d*.

Plain concrete arches have been used to a considerable extent in recent years, and have an advantage over the stone or brick masonry arches in that the structure is somewhat more elastic and may withstand tensile stresses to a slight degree although they should not be designed with this in view. In the design of such arches, as well as those of stone and brick, the line of pressure should fall within the middle third of the section, in order that no tensile stresses may be developed. If all the loads acting on the sewer were known exactly, it would be possible to design the section so that at no time would the line of pressure lie outside the middle third, but practically this is impossible, as our knowledge of the action of earth pressure is a matter of approximation only. On that account, under special conditions the stresses in the arch section may not be entirely due to direct compression, but in addition bending stresses may be developed.

Arches of reinforced concrete are not subject to the limitations just mentioned, but can be made to withstand heavy bending moments by reinforcing the section with steel bars to carry tensile stresses. In arches in which the line of pressure lies within the middle third, the stresses in the arch are mainly due to compression and the concrete must of necessity carry the principal part of the load, so that the steel cannot be stressed to the allowable limit. On the other hand, the presence of the steel reinforcement is of considerable value. Concrete is more reliable in compression than in tension, and on that account the steel furnishes a sort of insurance to the structure, to care for tensile stresses which may occur on account of unequal settlement of the foundations, temperature changes and many other conditions, of which the designer can have little knowledge. The steel is also an additional factor of safety against careless and defective construction. On account of its presence, it is possible to increase slightly the allowable working stresses in the concrete over those which should be used for plain concrete masonry. Because of these considerations the authors believe that for large sewer arches reinforced concrete offers greater advantages than plain concrete, even though an analysis of the section shows that the line of resistance for the conditions considered will remain within the middle third of the masonry section. An inspection of the analyses given in the following chapter will show how great a change may occur in the theoretical location of the line of resistance due to a change in the assumed conditions.

Electrolysis in Concrete.—Considerable study has been given recently to the corrosive effect of stray electric currents in concrete reinforced with steel. For a careful discussion of this subject, the reader is referred to "Technologic Paper No. 18" of the Bureau of Standards,

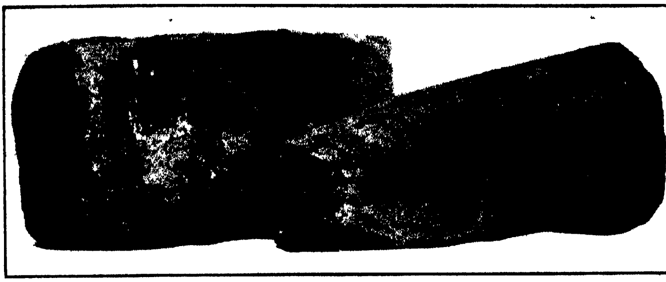


FIG. 174.—Brick from arch and invert of Worcester sewer.

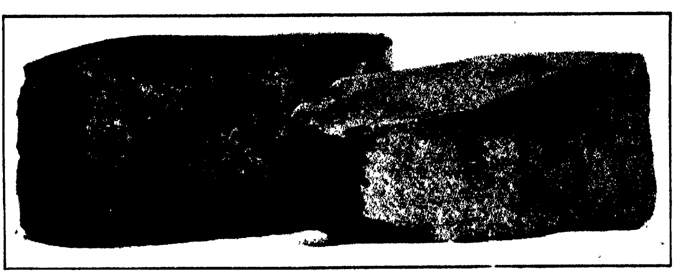


FIG. 175.—Brick from side of invert of Worcester sewer.

(*Facing page* 456)

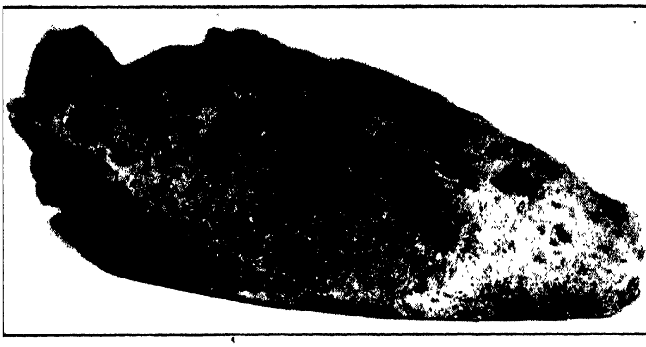


FIG. 176.—Brick from invert of Worcester sewer.

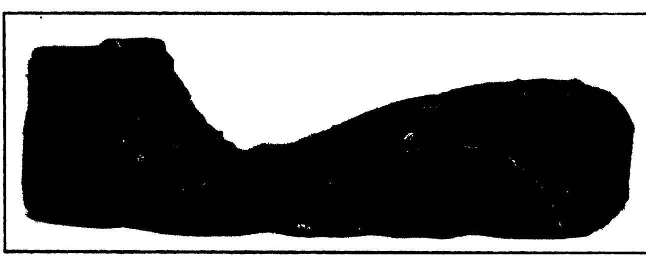


FIG. 177.—Brick forming manhole ledge.

(*Following Fig. 175*)

U. S. Department of Commerce, which also contains a bibliography of the subject.

Wear on Sewer Inverts.—A careful inspection made in 1909–10 of the condition of the brick sewers in Worcester, Mass., by the authors developed a number of interesting points. Many of these old brick sewers forming a part of a combined system of sewerage and varying in size from a 24 × 36-in. to a 48 × 72-in. egg-shaped section, were constructed between 1867 and 1880. Natural or Rosendale cement was used in nearly every case, and the majority of the sewers were built by contract.

The brick invert was found to be badly worn in all sections where the velocity flowing two-thirds full (Kutter's formula $n = 0.015$) exceeded 8 or 9 ft. per second. In some sections where the estimated velocity amounted to 12 or 13 ft. per second the first course of brick in the invert in places was worn through and the second course was partly worn. A majority of the streets are surfaced with gravel and during storms a large amount of street detritus washes into the sewers in spite of the many catch-basins. The effect of the scouring action of this material as it is swept or rolled along by the sewage can be seen on the brick which, especially below the dry weather flow line, were worn to smooth faces and rounded edges.

On slopes where the wear has been excessive it was quite generally true that the upstream ends of the brick were worn away more than the downstream ends. Figs 174 and 175 show brick from sewers at Worcester, Mass. The two in Fig. 174 were taken from a 30 × 45-in. egg-shaped section built by contract in 1874. The masonry of this sewer was constructed of two rings of sand-struck brick of 20 to 30 per cent. absorption, by volume, laid in Rosendale cement mortar. The brick shown were taken from a section where the grade is 0.0694. The velocity in this section, based on Kutter's formula, $n = 0.015$, at two-thirds full, is 22 ft. per second. The left brick was taken from the crown of the arch, on which there was no wear. The right brick was taken from the invert, the small end being the upstream end. The depth to which the mortar joints were washed out can be seen on the worn brick by the change in shade from dark to light, the light shade being caused by part of the mortar joint sticking to the brick. The mortar itself was very sandy and comparatively soft and little difficulty was experienced in removing the brick from the invert.

Fig. 175 shows two brick taken from a 48 × 72-in. egg-shaped section, built in 1872 by contract of 8-in. brickwork laid in Rosendale cement mortar. These two brick were taken from the side of the invert on a section where the grade was 0.0290 and the estimated velocity flowing two-thirds full was 20 ft. per second. The brick were exceedingly hard and dense, probably having an absorption of 8 to 12 per

cent., and were worn very smooth, almost to a polish. The small end in each case was the upstream end. The brick in the center of the invert were worn very much more than those shown, but owing to their excessive wear and consequent thinness and also on account of the depth of sewage, it was impracticable to remove any of them. In this section some of the first, or inner course brick, were worn through and the second or outside course was beginning to show wear.

Fig. 176 shows another brick taken from the same sewer and section as those shown in Fig. 174. This brick was laid in the invert in the position shown in the photograph. The right end was the upstream end. There was a bad hole in the invert at this point and the mortar was so completely washed out that the brick was removed with the fingers without the aid of a chisel. All that is left of one of the 4 × 8-in. faces is the little dark spot shown in the foreground at the left-hand end. The brick was somewhat below the average in quality and rather porous.

Fig. 177 shows a brick taken from the ledge or step, above the invert in a manhole constructed in 1868 by contract. The brickwork was laid in Rosendale cement mortar. In this manhole there were five inlet pipes which discharged surface water from several catch-basins and inlets; they were so located that in time of storm the flow from all five was concentrated in a 4- or 5-ft. drop to the brick ledge of the manhole. The force of the falling water and detritus wore a bowl-shaped depression in the ledge and side of the manhole. The left end of this brick shows its original thickness, being protected by the brick in the course above. This brick shows more clearly than can be described, the effect of the wearing action during a period of about 35 years. The next two brick adjacent to the one shown were worn even more and broke in pieces in removal owing to their extreme thinness. While this brick was not taken from a sewer invert, it shows very clearly the effect of even a small drop in the flow line and the resulting wear on the brickwork, such as might be expected from similar conditions in the invert.

The mortar joints were eroded to a much greater extent than the brick, which doubtless served to increase the wear on the brick, owing to the eddy currents caused by the additional roughness. This was not entirely due to the use of Rosendale cement, for the joints in the arch above the springing line were found to be in very good condition. Doubtless, some of the wear on the invert brick has been due to chipping action rather than abrasion.

In all cases where lateral sewers on steep grades entered well up from the invert, there were signs of considerable wear on the side of the main sewer where the stream from the lateral struck during times of storm

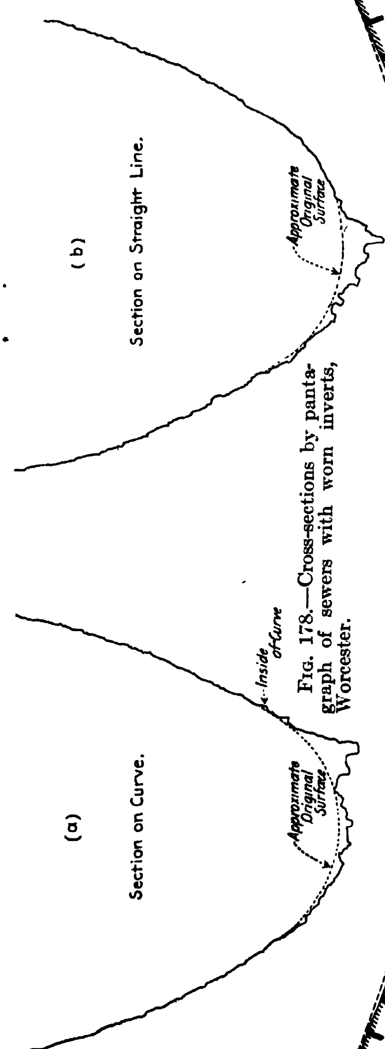


FIG. 178.—Cross-sections by pantagraph of sewers with worn inverts, Worcester.

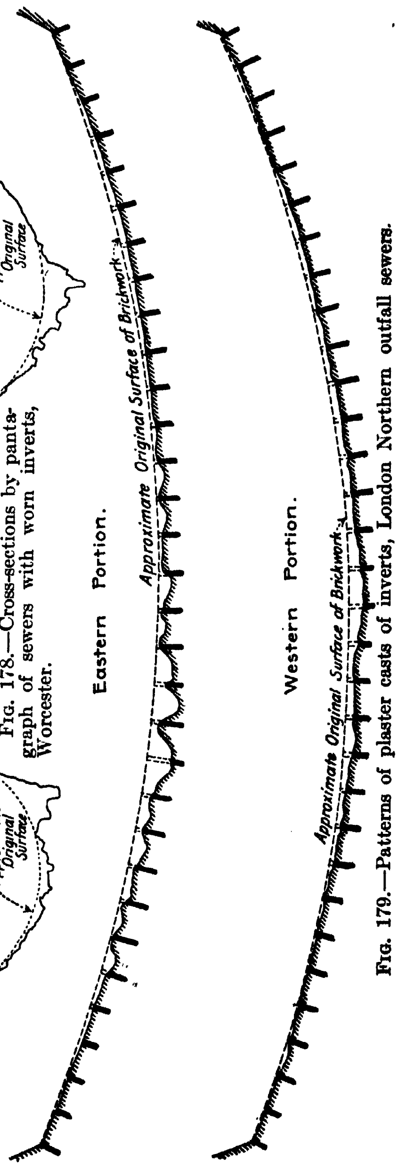


FIG. 179.—Patterns of plaster casts of inverts, London Northern outfall sewers.

flow. In drop manholes and other places where a fall of 4 or 5 ft. or more occurred, the brickwork under the drop was badly worn.

On curves constructed on grades producing velocities of 8 ft. per second or more, the brickwork on the inside of the curve was cut away, in several cases even through the second course of brick. A cross-section, Fig. 178a, of the interior surface of an egg-shaped section, 48 in. × 72 in., constructed by contract in 1872 of two rings of brick with Rosendale cement mortar on a grade of 4.32 ft. per 100 ft., shows this abrasion of the invert on a curve. Fig. 178b is a cross-section of the same sewer on a straight section. In each of the deep holes shown, the first course of brick had been worn away and part of the second. The dotted lines show the approximate original surface of the brickwork. These cross-sections were made by a specially constructed pantagraph. Soft brick were worn much more than hard brick, but where one soft brick was surrounded by hard brick, even these were worn more than a similar section where the brick were all hard.

On flatter sections of 100 to 200 ft. in length on either side of which were steep sections, there was some wear, due no doubt to the fact that the velocity in the flat section although not greater than 5 or 6 ft. per second theoretically, actually was much higher on account of the influence of the steeper sections above and below.

It is interesting to compare the experience gained at Worcester, with information obtained at Louisville, Ky., from an inspection of old brick sewers. Where the velocity was high, there was but little wear of the brick, while at Worcester sewers having apparently the same velocity showed serious wear. The explanation is that at Worcester the street detritus contains a large quantity of quartz sand coming from streets which for many years were, and to some extent still are, surfaced with gravel. There are also large deposits of sand and gravel in the city and the soil as a whole contains a large amount of quartz. In spite of the large number of catch-basins in use, considerable quantities of sand and gravel find their way into the sewers, and as the detritus is carried along by the flow of sewage the invert brick are worn by the harder material. At Louisville, the soil is composed of clay and disintegrated limestone and the streets are surfaced with crushed limestone, which, for the most part, is softer than the sewer brick. Even in sewers constructed of relatively soft brick, say those testing between 24 and 30 per cent. absorption, there appears to be but little wear from the velocities which at Worcester have caused serious wear. Although doubtless the detritus washed along the inverts at Louisville does cause some wear, the attrition is much more effective upon the detritus itself than upon the sewer brick.

Fig. 179 shows two patterns of plaster casts taken from the invert of one of the Northern outfall sewers, middle level, 9 × 9-ft. section,

leading to the Barking works, London, England. The upper pattern shows the eastern portion of the cast and the lower pattern shows the western portion. The dotted lines show the approximate original outlines of the brickwork and the approximate depth of wear can be judged by comparison with the thickness of the brick. The mortar joints are indicated by heavy black lines. The most interesting feature of these patterns is that they clearly show that the cement mortar in the joints was harder than the brick themselves, and resisted the wear longer than the brick did. This is exactly opposite to the experience in Worcester, Mass. This is the only instance which has come to the attention of the authors in which the mortar joints withstood the wear better than the brick. Although the old sewers in Worcester, Mass., were laid with Rosendale cement mortar, many of them have since been repaired with brick laid in Portland cement mortar and in many cases even these new inverts have shown considerable wear. It is possible, if these old sewers had been constructed in the first place with Portland cement mortar, that some such wear as that shown in Fig. 179 might have resulted although there are now no indications of such a result.

The full-size pattern from which Fig. 179 was made was furnished by John E. Worth, District Engineer of the London County Council. The casts were made April 14, 1897. Mr. Worth states that the reported relative condition of the brick and mortar was so unusual that plaster casts of the invert were made in order to verify and preserve the record.

Lining for Concrete Construction.—From the observations made and tests conducted by the authors it appears that on all slopes in which the estimated velocity of the sewage will be 8 ft. per second or greater, the invert may well be paved with hard burned or preferably vitrified paving brick with square edges, laid with the edges projecting as little as possible and with full Portland cement mortar joints. This invert paving should extend well up on the sides of the sewer, on straight sewers covering in general, the arc of an angle of 90 deg. at the center of a circular sewer. The use of brick paving, as above suggested, is preferable to concrete on account of the greater ease of making repairs and further on account of the probability that vitrified or even hard-burned brick will withstand the wear better than concrete of average quality. It is desirable when sewers are to be built of concrete to use hard aggregates, especially for inverts, and a first-class granolithic finish where the surface is subject to greatest wear is better than the ordinary concrete finish.

SURFACE LOADS TRANSMITTED TO SEWERS

Live Loads.—Sewers constructed in shallow cut are often subjected to the effect of loads on the surface, transmitted through the earth

filling. If the sewer line is crossed by steam railroad tracks there will be heavy loads from locomotives or loaded freight cars; if crossed by an electric railroad, there will be the loads from passenger or express cars,

TABLE 142.

STANDARD LOCOMOTIVE LOADINGS.

Cooper +		Axle Spacing, Ft	8'	5'	5'	5'	9'	5'	6'	5'		
Cooper +	Class E-30	Axle Load	15,000	30,000	30,000	30,000	30,000	19,500	19,500	19,500	19,500	Uniform Load 3,000 Lb per Lin Ft
Cooper +	Class E-40	Axle Load	20,000	40,000	40,000	40,000	40,000	26,000	26,000	26,000	26,000	Uniform Load 4,000 Lb per Lin Ft
Northern Pacific Ry *		Axle Load	25,000	52,000	52,000	52,000	52,000	36,000	36,000	36,000	36,000	Uniform Load 5,000 Lb per Lin Ft
Atchison, Topeka & Santa Fe Ry * Heavy Grade		Axle Spacing in Feet	7.5'	4.5'	4.5'	4.5'	10.5'	5'	5.5'	5'		
Atchison, Topeka & Santa Fe Ry * Heavy Grade		Axle Load	27,000	66,000	66,000	66,000	66,000	30,000	30,000	33,000	33,000	Uniform Load 4,800 Lb per Lin Ft

+ *From Cooper's General Specifications for Steel Railroad Bridges.*
* *From Trans. Am. Soc. C.E., Vol 54A, p 82.*

construction cars or snow plows, which in the case of the more recent high-class interurban lines amount to approximately the same as the loads on second-class steam railroads. In highways sewers are subject to the loads of steam road rollers, traction engines, and heavy trucks.

TABLE 143.

TYPICAL HEAVY FREIGHT CARS

Steel Coal Cars	Axle Spacing in Feet-Inches	5' 6"	19' 9"	5' 6"			
Steel Coal Cars	Axle Load	35,000	35,000	35,000	35,000		
Iron Ore Cars	Axle Spacing in Feet-Inches	5' 6"	17' 9"	5' 6"			
Iron Ore Cars	Axle Load	60,000	60,000	60,000	60,000		

From Trans Am Soc C.E., Vol 54 A, p. 85

For convenience in estimating live loads, four tables are given: Table 142, typical standard locomotive axle loads and spacings;

Table 143, typical axle loads of cars for heavy freight, such as coal or iron ore; Table 144, typical axle loads of the heavy type of electric cars for suburban service, and Table 145 the wheel loads and general dimensions of steam road rollers, traction engines and heavy automobile trucks. Fig. 180 shows the details of standard railroad track construction.

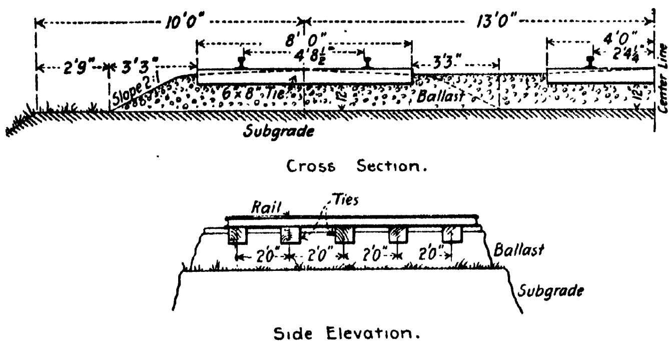
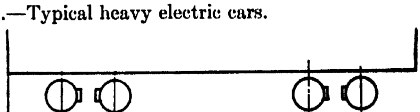


Fig. 180.—Standard railroad track construction.

Table 144.—Typical heavy electric cars.



Brooklyn Rapid Transit Co. 1907–31 Tons	Axle Spacing In Feet-Inches.	9'2½"	4'1"	15'11"	4'1"	9'2½"
	Axle Load	15,500	15,500		15,500	15,500
Long Island R.R. 1907–53 Tons	Axle Spacing In Feet.	5.45'	5.5'	27.9'	6'7"	5.45'
	Axle Load	30,800	30,800		22,200	22,200
New York Interborough Subway Trailer 1908–35 Tons	Axle Spacing In Feet-Inches.	4'11½"	5'6"	30'6"	5'6"	4'11½"
	Axle Load	17,500	17,500		17,500	17,500
Boston-Worcester St. Railway Co. 1906–50 Tons	Axle Spacing In Feet-Inches.	8'9⅝"	6'6"	22'10"	6'6"	8'9⅝"
	Axle Load	25,000	25,000		25,000	25,000
Boston Elevated Railway Co. Tunn. 1907–42 Tons	Axle Spacing In Feet-Inches.	8'4"	6'1"	16'11"	6'1"	8'4"
	Axle Load	21,000	21,000		21,000	21,000

From Jour. Assn. Eng. Soc. Dec. 1909 p. 241.

TABLE 145.—WEIGHTS OF STEAM ROAD ROLLERS

Data furnished by The Buffalo Steam Roller Co.

Rating	Total weight equipped in lb.	Load per wheel in lb.	Diameter of wheels in inches		Face width of wheels in inches		Distance c. to c. of axles ft. in.	Width of track in.
			Front	Rear	Front	Rear		
10 tons	26,000	8,670	44	69	47¼	18	9 10	
12 tons	31,000	10,340	46	69	51	20	10 8	
15 tons	39,000	13,000	48	72	52½	22	11 1	91
Weight of Traction Engine Data furnished by the Good Roads Mchy. Co.								
16 h.p.	19,580	6,530[1]	40	66	12	19	10 6	82
Weight of Typical Automobile Trucks								
5 tons	20,000[2]	6,900[1]	36	42	6	13	12 6	86

[1] Rear wheels.

[2] Allows for 25 per cent. overload.

The wheel loads from railroad rolling stock are well distributed over the road bed by the track, ties and ballast, so that for depths of earth fill of 5 ft. or more it is probably safe to estimate the axle loads as uniformly distributed over a somewhat larger area than that of the road

TABLE 146.—ESTIMATED INTENSITIES OF SURFACE LOADS

Loading	Estimated equivalent intensity of load on surface, lb. per sq. ft.	Assumed dimensions of loaded surface, ft.
Locomotive, Coopers E30	630	19×10
Locomotive, Coopers E40	840	19×10
Locomotive, Northern Pac. R. R	1,100	19×10
Locomotive, At., Top. & S. F. R. R	1,500	17.5×10
Steel coal car	740	9.5×10
Steel ore car	1,260	9.5×10
Electric car, Brooklyn R. T. Co	380	8.1×10
Electric car, Long Is. R. R	650	9.5×10
Electric car, N. Y. Interborough	370	9.5×10
Electric car, Boston and Worcester	470	10.5×10
Electric car, Boston Elevated	410	10.1×10
	lb. per. lin. ft. of trench[1]	
Steam road roller, 10 tons	8,670	....
Steam road roller, 12 tons	10,340	....
Steam road roller, 15 tons	13,000	....
Traction engine, 16 H.P.	6,530	....
Automobile truck, 5 tons	6,900	....

[1] Assuming weight of one wheel per linear foot of trench. If trench is wide enough to receive both rear wheels load assumed should be that upon the two rear wheels.

bed directly under the loads. For locomotives, then, the heaviest concentration would occur under the driving wheels, or in the case of freight or passenger cars, under one truck. In Table 146 are estimates of the intensity of such loads on the ground surface.

The loads from the wheels of steam road rollers, traction engines, trucks, etc., are applied directly to the surface of the fill but over a very small area. Although the intensity of the load at the surface is great, it becomes distributed fairly well over the entire width of the trench for depth of 5 ft. or more and in a similar manner longitudinally. In such cases the maximum load of a single wheel may be estimated as distributed over 1 lin. ft. of trench surface, and further distributed longitudinally depending on the depth.

Where the crown of the sewer is not more than 5 ft. below the surface, an increased load may be assumed on account of the impact and vibration caused by swiftly moving trains or cars. In the case of express trains moving at a high speed this impact may possibly produce a load or blow 50 per cent. greater than the load when not in motion.

Dead Loads.—In manufacturing districts, sewers are often subjected to heavy surface loads from piles of lumber, brick, pig iron, coal, etc. Wherever such is likely to be the case ample allowance should be made. It is not uncommon to find surface loads as high as the following: lumber, 850 lb. per square foot.; brick, 900 lb.; coal, 1200 lb.; and pig iron, 2300 lb.

There are cases, doubtless, where heavy masonry foundations have been built over sewers without regard for their stability. Wherever it is necessary to do such work, either the sewer arch should be strengthened to carry the excess load, or preferably the foundation in question should be built so as to relieve the sewer arch of all of the load of the building or structure.

Proportion of Loads Transmitted to Sewers.—The best information available (in 1914) as to the weight of superimposed loads or surface loads transmitted to sewers will be found in Bulletin 31, Engineering Experiment Station, Iowa State College, "The Theory of Loads on Pipes in Ditches," by A. Marston and A. O. Anderson. An abstract of part of this work will be found in Chapter X, on Sewer Pipe. In Fig. 181 are plotted curves of the values of C in the formula $L_p = CL$, where L_p is the total load per unit of length on the sewer, C is a coefficient in which allowance is made for the ratio of the width and depth of trench, for the friction of the backfill against the sides of the trench, and for the character of the backfilling material; L is the surface load per linear foot of trench; B is the width of the trench at the top of the pipe, and H is the height of fill in the trench above the top of the sewer.

By "long loads" are meant those which extend a long distance along the trench as compared with its width and height. In this class come such loads as those resulting from piles of brick, lumber, pig iron, coal,

etc., and possibly in freight yards from long lines of cars on storage tracks.

By "short loads" are meant such as those from road rollers, trucks or wagons, and in general, all of the other "live" loads previously mentioned.

The curves in Fig. 181 will be found of value in estimating the proportion of the weight of surface loads that might be transmitted through the backfilling to the sewer. All such loads, after having been reduced in the proportion shown by the curves or as aided by judgment, should then be changed to the basis of an equivalent earth load, for convenience in designing. By this means, the backfilling and surface loading will be reduced to the same relative unit weights and can be considered together as a certain total depth of backfill.

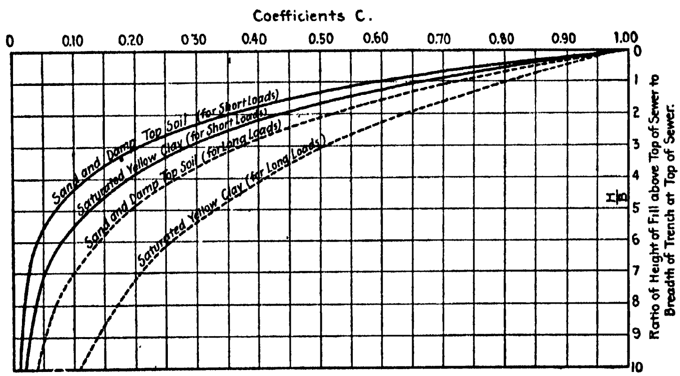


FIG. 181.—Coefficients of surface loads transmitted through earth fill to sewers.

For example, the depth of earth over the crown of a certain sewer is 20 ft. and the width of trench at the top of the sewer is 10 ft. The backfilling material is sand weighing 120 lb. per cubic foot. One section of this sewer is to be built under a coal yard, and accordingly there should be added a surface load due to piles of coal of 1200 lb. per square foot. The total "long load" per linear foot of trench, L, would be $1200 \times 10 = 12{,}000$ lb. The ratio of height of fill to width of trench, $H/B = 2$. On Fig. 181, follow along the horizontal line $H/B = 2$ until it intersects the curve for sand and damp top soil for long loads, which point is on the vertical line (interpolated) for coefficient $C = 0.52$. Substituting in the formula $L_p = CL$ the values of $C = 0.52$ and $L = 12{,}000$, we have $L_p = 0.52 \times 12{,}000 = 6240$ lb. per linear foot of sewer.

Since the width of trench, B, is 10 ft. and the assumed unit weight of trench filling is 120 lb., the equivalent earth load of $L_p = 6240$ lb. per linear foot is $6240 \div (10 \times 120) = 5.2$ ft. of sand backfilling.

WEIGHT OF BACKFILLING

For most designing work, it is sufficient to assume that the backfill will weigh 100 lb. per cubic foot and that the horizontal pressure at any depth due to this fill will be one-third of the vertical pressure. Where more nearly exact assumptions must be made, the material which will be used should be actually weighed, in a moist as well as a dry condition, or the information given in Table 108, page 334, should be employed. The weight of the whole of the backfilling is not transmitted to the sewer in most cases, but only a part, which may be estimated from Table 109, page 335, and its accompanying explanatory text. It gives somewhat lower pressures than Rankine's formula, explained in Volume II, and for this reason some engineers are inclined to defer its use until experience has shown that it is safe to employ these lower pressures.

Rankine's Theory.—When the Rankine theory is used in designing sewer arches, the surface of the earth is usually assumed to be horizontal. The total earth pressure acting on a section of a sewer arch may be considered as composed of a vertical component equal to the weight of the column of earth above the section, and a horizontal component which at any point cannot be greater than $(1 + \sin \phi)/(1 - \sin \phi)$ times the vertical pressure at the same point, nor less than $(1 - \sin \phi)/(1 + \sin \phi)$ times the vertical pressure, ϕ being the angle of repose. The former expression represents the passive resistance of the earth, while the latter represents the active pressure which, at least, is probably realized. If the angle ϕ is taken as 30 deg., which is a convenient figure to use and probably represents average conditions, the above statement means that the horizontal pressure cannot be greater than three times the vertical pressure nor less than one-third of it. While it is recognized that a more logical course would be to use exact values for the angle of repose, or better, the angle of internal friction, this is hardly justified for average conditions because of the great uncertainty regarding the action of earth pressures and the variation in the character and condition of trench materials.

For trenches in which the sheeting will be left in place up to within a few feet of the surface, care should be taken in using formulas similar to the above, on account of the fact that the sheeting introduces different conditions and prevents cohesion between the backfilling material and the sides of the trench, which might otherwise exist. It is probable that in close-sheeted trenches newly backfilled, almost the entire weight of the backfill may come directly on the sewer structure. On this account it is a question whether a designer is justified in reducing the dead weight of earth transmitted to the sewer by any such methods as those just described. It is often impossible to tell in advance whether sheeting will be left in place or not. In the practice of the authors it

has been customary to assume that the entire weight of the earth fill will be transmitted to the sewer, even though it is recognized that in many cases no such severe loading is encountered. On the other hand the actual weight of superimposed loads transmitted to the sewer may very well be reduced in the manner suggested.

Mohr's Method of Determining Pressures.—A graphical method of determining earth pressures, devised by Prof. Mohr in 1871 and founded on Rankine's theory, was described by Prof. G. F. Swain in the "*Journal" of the Franklin Institute*, vol. cxiv, p. 241. It is as follows:

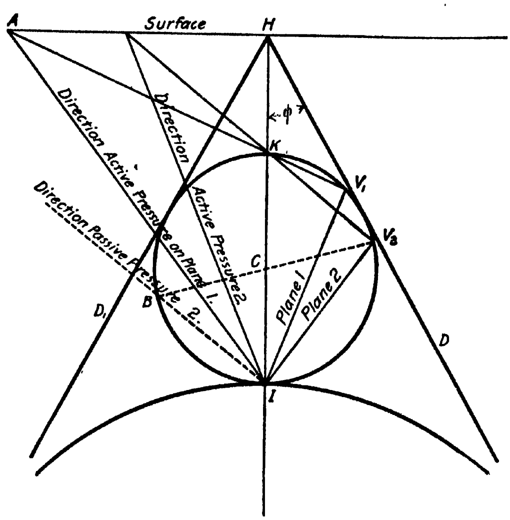


FIG. 182.

Let a horizontal line AH Fig. 182 represent the surface of the earth. Draw HI perpendicular to AH, and of some convenient length, as 5 in., equivelent to 10 ft. on a scale of ½ in. to 1 ft. Lay off HK of length.

$$HK = HI \tan^2(45° - \tfrac{1}{2}\phi)$$

ϕ = angle of repose. This will be recognized as equivalent to Rankine's formula for the intensity of earth pressure, with w, the unit weight of earth, omitted.

$$P = wh \tan^2(45° - \tfrac{1}{2}\phi)$$

where P = the intensity of earth pressure by Rankine's theory and h = the depth of earth.

Having located point K, with KI as a diameter, describe a circle. Through I draw a line IV_1 parallel to the face of the wall or section of arch upon which the pressure of the earth acts. Draw V_1K through the points V_1 and K on the circumference of the circle, and prolong it to meet the surface line AH. At this point of intersection A, draw AI, which gives the direction of the active pressure on plane 1. The distance HV_1 measures the magnitude of this pressure for the depth represented by HI. $(HV_1/HI)w$ is the intensity of active pressure per unit depth of earth on plane 1. The magnitude of HV_1 can be obtained by scaling the line HV_1. In a similar manner the direction and amount of the active pressure on any other plane, as plane 2, can be found.

The amount of the maximum passive earth pressure is measured by HI for a depth of HV_1 ($HI/HV_1=$ intensity) for plane 1, or by HI for a depth of HV_2 for plane 2. The direction of the maximum passive pressure is found by drawing through V_1, V_2, etc., a diameter of the circle, and then connecting the point of intersection B with I. Line BI is the direction of the maximum passive pressure for plane 2. It is perpendicular to the face upon which pressure is exerted.

There is an exact mathematical proof of the foregoing, but the following general proof will probably be sufficient.

If, in the figure we let the line HI represent a vertical plane, we have chosen HK of such a distance that for the depth HI, HK represents the intensity of the active earth pressure.

It can be proved that as the plane of the wall slants away from the vertical, a circle of diameter KI will contain all the points V for every position of the plane, the intensity being HV/HI until a horizontal surface is reached which has a pressure of $HV/HI = HI/HI = 1$, or the total dead weight of the earth above the plane. The angle IHV is the angle S, or the angle which the stress makes with the normal to the plane.

From Rankine's theory we know that the angle S can never exceed the angle of friction ϕ, or the angle of repose of the earth. Hence, if we draw from H two lines making angles of ϕ on either side of HI, we know the circle must lie within those lines, and when the earth is just on the point of slipping $S = \phi$ and the circle is tangent to the two lines HD and HD'. There are two circles which satisfy the conditions representing the two limiting states of equilibrium when the earth is just ready to slip. The larger circle, only part of which is shown in Fig. 182, represents the case where the maximum principal pressure HI is increased until the limiting condition is reached. This is the passive earth pressure. The smaller circle represents the case where the minimum principal pressure HK is decreased until the limiting condition is reached. This is the active earth pressure. In the case of $\phi = 30°$, for which the figure is drawn, the passive earth pressure is 9 times the

active. It is not necessary, however, to use the large circle, since for the active pressure

$$P_a = wh\,\frac{1 - \sin\phi}{1 + \sin\phi}$$

and for the passive presssure

$$P_p = wh\,\frac{1 + \sin\phi}{1 - \sin\phi}$$

the term $(1 - \sin\phi)/(1 + \sin\phi)$ being merely inverted. The inversion has been accomplished as follows:

The active pressure per unit depth $= w(HV/HI)$

The passive pressure per unit depth $= w(HI/HV)$

The angle $IHV =$ the angle S, the angle between the normal to the plane and the stress. Therefore, having this angle, we can erect a normal to the plane and lay off the angle S, thereby obtaining the direction of the stress. For example:

$$\text{angle } IAV_1 = \text{angle } IHV_1$$

CHAPTER XIII

THE ANALYSIS OF MASONRY ARCHES

There are a number of methods in use today for analyzing the stresses in arches. While a considerable proportion of existing large masonry sewers have been designed without any analysis of the stresses, the increasing use of reinforced concrete sewers is responsible for a more general effort on the part of designers to analyze the stresses in these structures.

In the following pages, three methods of analysis are described and the diagrams and computations for each applied to a 15-ft. 6-in. × 15-ft. 2-in. horse-shoe sewer section are given.

The first method, called the "voussoir arch method," based on the so-called "hypothesis of least crown thrust," is applicable only to that portion of the sewer section above the springing line of the arch. Either the sewer must have very heavy side walls or the thrust of the arch must be carried by the sides of a rock trench in order to make this method strictly applicable.

The second method, based on the elastic theory of the arch and following the method described by Turneaure and Maurer in their "Principles of Reinforced Concrete Construction" is applicable to all sewer sections and can be used to cover all conditions. It has some mechanical disadvantages when applied to the analysis of the entire sewer structure, invert included.

The third method, also based on the elastic theory but using the so-called method for indeterminate structures, is of special advantage in the analysis of the entire sewer section as it permits a more suitable division of the axis in the side wall and invert. It does, however, require some additional labor over the second method when applied to an arch with fixed ends. For large sewers constructed in compressible soil and built of monolithic reinforced concrete, the third method is the most desirable.

Attention is particularly called to the fact that in the following analyses the terms "elastic theory" and "method for indeterminate structures" are used merely to distinguish between the two methods, both of which are based on the elastic theory and are applicable to indeterminate structures. The practical difference between the two is in the method of subdividing the arch axis.

Since the three examples given are based so far as practicable on the same assumptions, a direct comparison may be made of the results obtained.

Another method has been used by some engineers, that of Prof. Chas. E. Greene for an arch rib with fixed ends. Reference "Trusses and Arches" Part III, by Greene. According to W. W. Horner, Principal Assistant Eng., St. Louis Sewer Department, this method was used for the earlier arches designed under his direction. Later, it was worked up in the form of general formulas for each 10-deg. point on the arch. Similar formulas have been

published by A. E. Lindau, *Trans. Am. Soc .C. E.* vol. lxi, 1908, p. 387. Mr. Horner stated that "this method is satisfactory where the arch rests on rock or on a heavy invert, but the introduction of a side wall of over a foot in height causes the whole structure to depart somewhat from fixture at the spring line." He further states that Greene's method was used in 1914 to give a trial section for all larger arches or work of especial importance and that the work was checked by the elastic theory method of Turneaure and Maurer.

ANALYSIS OF ARCH BY VOUSSOIR METHOD

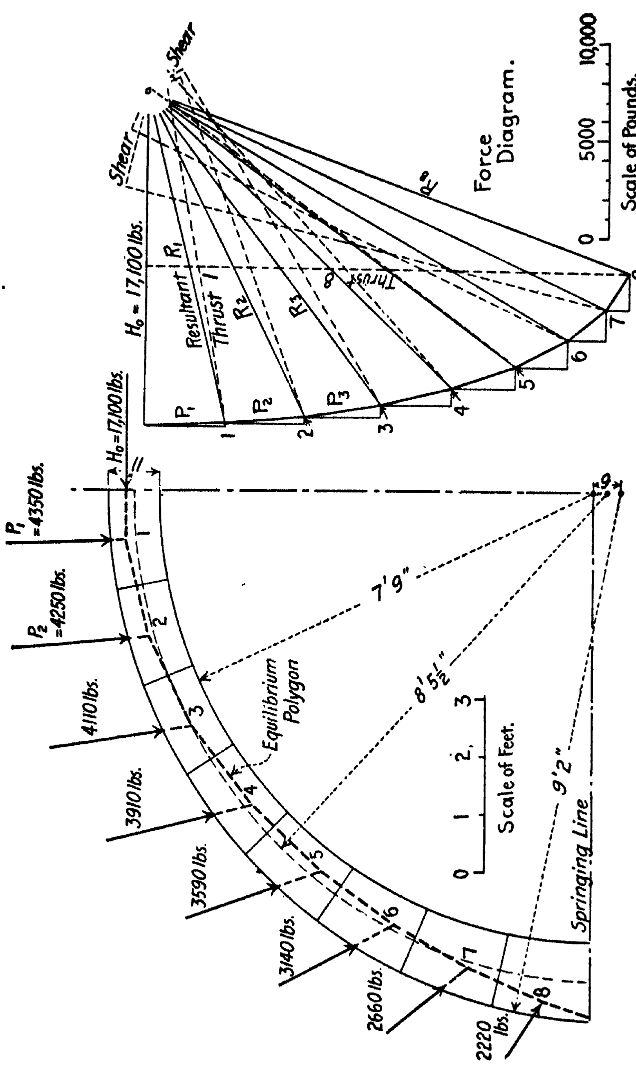
Masonry arches may be divided into two general classes, voussoir and monolithic arches, the former constructed of separate stones or bricks, while the latter are monolithic. In designing concrete arches, they may be considered as composed of a number of sections, in which case they come under the voussoir classification. Sewer arches may be further classified as hingeless, that is, with fixed ends.

There are a number of theories on which the design of voussoir arches has been based, but the one most generally employed is the rational theory, based on the hypothesis of least crown thrust. The following application of this theory to the design of sewer arches is based on a discussion in Baker's "Masonry," tenth edition, page 620.

According to the hypothesis of least crown thrust, the true line of resistance of the arch is that for which the thrust at the crown is the least possible in amount consistent with the arch being in a state of equilibrium. This theory asssumes that the external forces acting on the arch create a thrust at the crown sufficient in amount to establish equilibrium in the arch, and that when this state of equilibrium has been established there is no need for further increase in the amount of this thrust and that therefore the thrust is the least possible consistent with equilibrium. These assumptions do not of themselves locate the line of resistance, but if the external forces are known in amount and direction, and the direction of the thrust is assumed, sufficient data will be provided to locate the line of resistance corresponding to the least possible crown thrust. The rational method assumes that the earth pressure acting on the arch is composed of vertical and horizontal components.

The direct determination of the line of resistance for an arch unsymmetrically loaded is practically impossible under this theory. As a general rule, however, sewer arches may be considered as being symmetrically loaded, and the following example is based on this assumption.

Let us assume that it is desired to locate the line of resistance of the 15-ft. 6-in. span concrete arch shown in Fig. 183, the relative thickness of the arch having been assumed, either with the aid of some of the empirical formulas previously given in earlier chapters, or in the light of experience with arches already constructed. As it has been assumed that the arch is symmetrically loaded, but half of the arch section need be drawn, as shown in the figure. Assume that the arch supports a depth of earth of 24 ft. above the crown, and that the unit weight of earth is 100 lb. per cubic foot and the unit weight of the concrete masonry 150 lb. per cubic foot.



Arch Section and Equilibrium Polygon.

FIG. 183.—Analysis of semicircular arch by the voussoir method.

In order to simplify the computation the design will be based on a section of the arch ring 12 in. thick, perpendicular to the plane of the paper. Also divide the center line of the half-arch section into eight equal parts, separated by radial lines as shown, to serve as the theoretical voussoirs for analytical purposes.

Vertical Forces.—The vertical forces acting on the arch section are the weight of the concrete section and the weight of the column of earth above the section. The weight of the concrete section acts through the center of gravity of the section, which for practical purposes may be assumed at the center line of the arch for that section. The weight of the earth prism above the arch may be assumed to act through the center of the horizontal projection of the extrados of the section. The center of gravity of the combined vertical load, that is, the weight of the concrete plus the weight of the earth, can be determined by moments, either analytically or graphically. The value of the weight of concrete, the vertical earth pressure and the sum of these two are given for each section in Table 147.

TABLE 147.—COMPUTATIONS OF FORCES

1 Section number	2 Weight of concrete, lb.	3 Vertical earth pressure, lb.	4 Total vertical force, lb.	5 Horizontal earth pressure, lb.	6 Resultant force on section, lb.
1	230	4120	4350	140	4350
2	230	4000	4230	390	4250
3	240	3820	4060	650	4110
4	250	3540	3790	920	3910
5	270	3120	3390	1170	3590
6	290	2510	2800	1430	3140
7	310	1780	2090	1650	2660
8	330	890	1220	1850	2220

Horizontal Forces.—Following the suggestion in regard to the intensity of the horizontal earth pressure given in a previous paragraph, if we assume the angle of repose equal to 30 deg., the intensity of the horizontal earth pressure will be one-third of the intensity of the vertical pressure at that point. The values of the horizontal earth pressures computed in this manner for each section, are given in Table 147.

Crown Thrust.—The section of the arch shown in Fig. 184 is held in equilibrium by the vertical forces due to the sum of the weights of concrete and earth prism, by the reaction R at the springing line or abutment and by the thrust T at the crown. The direction of the reaction at the abutment is immaterial in this discussion. Let H_o = the thrust at the crown; x_1 = the horizontal distance from the point of application of the reaction on the abutment to the line of action of w, representing the total vertical force on the first section of the arch from the crown; x_2 = the same for w_2; etc.; y = the vertical distance from the point of application of the abutment reaction to the line of action of H_o, the thrust on the crown; K_1 = the perpendicular distance from the point of application of the abutment reaction to the line

of action of h_1, the horizontal force acting on the first section of the arch; K_2 = the same for h_2; etc. Then by taking moments about the point of application of the abutment reaction we have the following equations:

$$H_o y = w_1 x_1 + w_2 x_2 + \text{etc.} + h_1 K_1 + h_2 K_2 + \text{etc.}$$

From this we obtain,

$$H_o = \frac{\Sigma wx + \Sigma hK}{y}$$

From the above equations it appears that, other things remaining the same, the larger y the smaller H_o, and therefore, in order to obtain a minimum value of the thrust H_o, the point of application of the thrust at the crown should be as near the extrados as is possible without stressing the masonry too high. It is usually assumed that the thrust acts at a point one-third of the depth of the arch from the extrados at the crown. This assumption means that the unit compressive stress at the crown is equal to twice the thrust H_o divided by the thickness of the arch at the crown, the length of arch section being considered as unity, which has already been assumed.

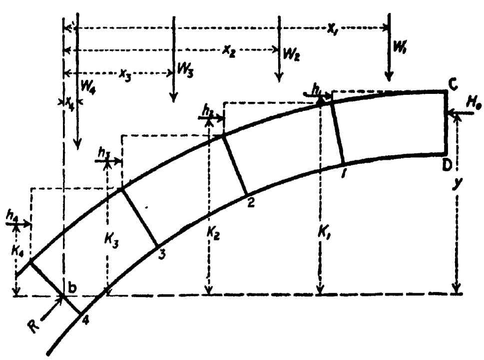


Fig. 184.

It is also usually assumed that the thrust is horizontal in direction. If the arch is symmetrically loaded this assumption is a reasonable one, but for conditions where the arch is unsymmetrically loaded the thrust at the crown cannot be horizontal and on that account a direct determination of the line of resistance by this method is impossible.

Joint of Rupture.—The value chosen for the crown thrust H_o must be such that the semi-arch will be in equilibrium. If H_o is too small some one of the joints may open at the extrados, and on the other hand, if H_o is too large, some one of the joints may open at the intrados. It is evident that neither of these conditions would result in a condition of equilibrium under the assumption that the center of pressure or line of resistance must remain within the middle third of any joint. If the line of resistance is so located, there will be no tension on any of the joints and consequently there will be no opening of any of the joints, either on the extrados or intrados. The required value of H_o

TABLE 148.—COMPUTATIONS OF MOMENTS AND CROWN THRUSTS

Origin of moments at section	Arms of vertical forces in feet								Arms of horizontal forces in feet								y ft.	Σwx ft. lb.	ΣhK ft. lb.	Total crown thrust $\frac{\Sigma wx + \Sigma hK}{y}$ lb.
	x_1	x_2	x_3	x_4	x_5	x_6	x_7	x_8	K_1	K_2	K_3	K_4	K_5	K_6	K_7	K_8				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
1	0.74								0.69								0.47	3,220	100	7,060
2	2.26	0.58							1.15	0.82							0.92	12,300	489	13,900
3	3.66	1.98	0.39						1.88	1.57	0.95						1.67	25,900	1,490	16,400
4	4.90	3.22	1.63	0.19					2.89	2.58	1.96	1.05					2.68	42,200	3,660	17,100
5	5.91	4.23	2.64	1.20	−0.03				4.12	3.81	3.19	2.28	1.10				3.89	58,800	7,510	17,000
6	6.69	5.01	3.42	1.98	0.75	−0.25			5.51	5.20	4.58	3.67	2.49	1.09			5.29	73,500	13,600	16,500
7	7.17	5.49	3.90	2.46	1.23	0.23	−0.48		7.02	6.71	6.09	5.18	4.00	2.60	1.02		6.80	83,300	22,400	15,500
8	7.37	5.69	4.10	2.66	1.43	0.43	−0.28	−0.61	8.57	8.26	7.64	6.73	5.56	4.16	2.58	0.87	8.36	87,400	33,900	14,500

will therefore be that value which will keep the line of resistance within the middle third of every joint.

If b, the origin of moments or point of application of the reaction at any joint, as in Fig. 184, be taken successively at the inner or lower end of the middle third of each joint, the corresponding value of H_o will be the crown thrust for which that particular joint is on the point of opening at the extrados; and if under this condition the greatest value of H_o that will prevent any joint from opening on the intrados be found, then that value is the crown thrust required by the hypothesis, for a smaller value will permit one or more joints to open at the extrados and a greater value will cause one or more joints to open at the intrados.

The joint for which the tendency to open at the intrados is the greatest is called the joint of rupture. Practically speaking, the joint of rupture is the true springing line of the arch and the rest of the arch below the joint of rupture is in reality a part of the abutment or side wall. The next step in the arch analysis is therefore to find the joint of rupture.

Crown Thrust for Joint Rupture. The total vertical forces as computed are given in Table 147. The moment arm of each of these forces or weights, with reference to the several origins of moment, is measured and entered in Table 148. In Table 148 are columns headed "Arms of Vertical Forces" and to the right of these a series of columns headed "Arms of Horizontal Forces." In column 2 will be found 0.74, which is the perpendicular distance or moment arm of the force, or weight, w_1, Fig. 184, about the lower middle third point of joint 1. Similarly, 2.26 is the arm of w_1 about the lower middle

third point of joint 2; 3.66 about the lower middle third point of joint 3; 4.90 about the lower middle third point of joint 4. In the same manner 0.58 in column 3 is the arm of the weight, w_2 about the origin of moments or lower middle third point of joint 2.

The horizontal forces as computed are given in Table 147. In a similar manner as described above, the arms of the horizontal forces, h_1, h_2, etc., are scaled and entered in Table 148. The moment arms of these horizontal forces denoted as K_1, K_2, etc., are shown in the tables. For example, under column 10, is 0.69 the perpendicular distance from the horizontal force h_1 to the lower middle third point of joint 1; 1.15 is the perpendicular distance from the horizontal force h_1 to the lower middle third point of joint 2, and so on.

The value of y, the moment arm of the crown thrust, is found by scaling the drawing, Fig. 183, and is recorded in Table 148. For example, 0.47 in column 18 is the perpendicular distance from the crown thrust, assumed to be applied at the upper middle third of the crown joint, to the lower middle third point of joint 1, and so on for the other values to the origin of moments of the several joints.

The next step is to find the sum of the moments of all of the vertical forces to the left of each of the origins of moments of the various joints, that is, for joint 1, the moment of the vertical force at the left of that joint equals w_1x_1; for joint 2, the moment of the vertical forces equals $w_1x_1 + w_2x_2$ and so on for each of the other joints. The moments of the vertical forces about each of the joints thus found are recorded in column 19.

In a similar manner, find the moment of the horizontal forces about each joint and record the sum for each joint in column 20.

The total crown thrust for each joint is then found by adding the moment due to the vertical forces and the moment due to the horizontal forces and dividing by the larger arm, y, of the crown thrust.

$$H_o = \frac{\Sigma wx + \Sigma hK}{y}$$

The value of the crown thrust thus obtained is recorded in the column 21 of Table 148. An inspection of this column shows that the crown thrust for joint 4 is the greatest and therefore joint 4 is the joint of rupture.

Force Diagram.—The maximum crown thrust for the joint of rupture has already been found as 17,100 lb. To construct the force diagram, a horizontal line is drawn to scale, see Fig. 183, to represent the amount of the maximum crown thrust as found for joint 4. This may be drawn at any convenient scale, as 1 in. = 3,000 lb. From the left end of the horizontal line lay off w_1, the first vertical force, vertically downward and from its extremity lay off h_1 horizontally to the right. Then the line from the right extremity of h_1 to upper end of w_1 represents the direction and amount of the resultant external force, P_1 acting upon the first division of the arch ring. The line R_1 drawn from the right extremity of h_1, or the lower extremity of P_1 represents the resultant pressure of the first arch stone upon the one next below it. Similarly, lay off w_2 vertically downward from the right extremity of h_1 and lay off h_2 horizontally to the right; then a line P_2 from the upper end

of w_2 to the right end of h_2 represents the resultant of the external forces acting on the second division of the arch, and a line R_2 from the lower extremity of P_2 represents the resultant pressure of the second arch stone on the third. The force diagram is completed by drawing lines to represent the other values of w, h, P and the corresponding reactions. The broken line P_1, P_2, P_3, etc., is sometimes called the "load line," as it represents the external forces acting on the arch in direction and, by scale, in amount in the order of their application to the arch, starting from the crown and going toward the springing line. The radial lines from the several points on this load line to the right end of the horizontal line are called the "rays" and represent in direction and amount the successive reactions or thrusts of one arch stone against the next section below.

Line of Resistance.—On the arch section through the several points of application of the horizontal and vertical forces, draw the resultant forces acting on each arch section. These may be taken from the force diagram.

To construct the line of resistance, draw through the upper limit of the middle third of the crown joint a horizontal line to an intersection with the oblique force P_1 acting on section 1; and from this point draw a line parallel to R_1 and prolong it to an intersection with the oblique force P_2 acting on section 2 of the arch. In a similar manner continue to the springing line.

The intersection of the line parallel to R_1 from the force diagram with the first joint gives the center of pressure on that joint; and the intersection of R_2 with the second joint gives the center of pressure for that joint and so on for the other joints.

On account of the method used, the line of resistance must pass through the lower middle third point of the joint of rupture. This offers a reliable method of checking the accuracy of the work of drawing the line of resistance.

The equilibrium polygon gives the resultant pressure acting on each joint. The thrust, normal to the joint, and the shear can be formed by resolving the resultant pressure into its two components tangent and perpendicular to the arch axis at the point in question. The values may be obtained by scaling those lines shown broken in the force diagram Fig. 183.

Having given the location and amount of the thrust on each joint, the stresses for that joint can be computed, as will be explained in a later paragraph.

ANALYSIS OF ARCH BY ELASTIC THEORY

The method of analysis of an arch section, based on the elastic theory, assumes that the arch is held in equilibrium by its resistance to combined compression and bending, that is, the arch is considered as a curved beam. This method is applicable to all hingeless arches of variable moment of inertia and to any system of loading, although the work is greatly simplified when the loads are symmetrical. As a rule, sewer arches can be considered as being symmetrically loaded.

For a more complete discussion of the theories and methods of analysis than is here given the reader is referred to "Symmetrical Masonry Arches,"

by Prof. M. A. Hôwe, a "Treatise on Masonry Construction," by Prof. Ira O. Baker, "Concrete, Plain and Reinforced," by Taylor and Thompson, or "Principles of Reinforced Concrete Construction," by Turneaure and Maurer. The method here given is that explained by Turneaure and Maurer.

The analysis of an arch consists of the determination of the forces acting at any section, usually expressed as the thrust, the shear and the bending moment at such sections. The thrust is taken to be the component of the resultant parallel to the arch axis at the given point and the shear is the component at right angles to such axis. The thrust causes simple compressive stresses, the shear causes stresses similar to those produced by the vertical shear in a simple beam.

In the analysis, the length of the arch will be considered as one unit perpendicular to the plane of the figure.

Let H_o = thrust at the crown,
V_o = shear at the crown,
M_o = bending moment at the crown, assumed as positive when causing compression in the upper fibers,
N, V and M = thrust, shear, and moment at any other section,
R = resultant pressure at any section = resultant of N and V,
ds = length of a division of the arch ring measured along the arch axis,
n = number of divisions in one-half of the arch,
I = moment of inertia of any section = I (concrete) + nI (steel) where $n = 15 = E_s/E_c$,
w, h, P = the vertical, horizontal and resultant external loads on the arch, respectively,
x, y = co-ordinates of any point on the arch axis referred to the crown as origin. All positive in sign,
m = bending moment at any point in the half arch section, Fig. 186, due to external loads. All negative in sign.

For symmetrical loads, the following equations can be derived:

$$H_o = \frac{n\Sigma my - \Sigma m\Sigma y}{(\Sigma y)^2 - n\Sigma y^2}$$

$$M_o = -\frac{\Sigma m + H_o\Sigma y}{n}$$

$$V_o = 0$$

The above calculations are for the half arch section.

The total bending moment at any section

$$M = m + M_o + H_o y$$

In the following analysis based on the elastic theory and using Turneaure and Maurer's method, two cases are considered.

Case I (Fig. 185).—In this case the invert is considered as being separated from the side walls and arch, and the elastic structure to be analyzed consists only of the side wall and arch section. This assumes that the base

of the side wall is fixed, that is, the arch is hingeless. Such a condition would exist where the sewer is constructed in rock cut with the base of the side walls or the invert resting on ledge rock. If the rock extended to a point above the springing line or horizontal diameter of the semi-circular arch, the analysis might properly be confined to that portion of the structure above the springing line of the semi-circular arch, as the ends could then be considered as fixed at that point.

Case II (Fig. 186).—This case differs from the preceding in that the entire structure, invert included, is considered as an elastic monolith and consequently subject to direct stress and bending at any point. Such a condition will be reached if the sewer is constructed in compressible soil and acts as ring. Reinforced-concrete sewers constructed in sand, gravel, or clay without special foundations should be treated under this case.

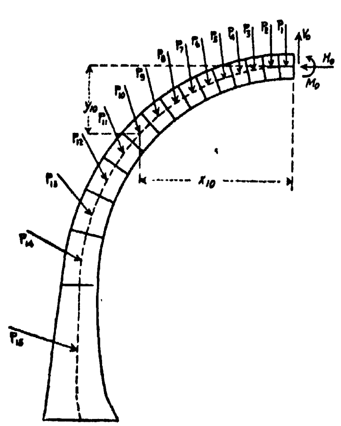


Fig. 185.

Analysis of Case I.—In the following discussion, the term arch is used to denote that portion of the section from the crown to the base of the side wall or the beginning of the invert. The half-arch section is drawn to some convenient scale, which should be sufficiently large to enable all distances to be scaled without appreciable error. The half-arch section under consideration is shown in Fig. 186.

Division of Arch Ring to Give Constant ds/I.—The first step in the analysis is to divide the half-arch section into a number of divisions of such length that the ratio of ds/I will be constant for each section. The following method of determining the successive divisions of the arch is taken from Baker's "Masonry," 10th edition, p. 676. While there are a number of other methods which may be used, this is one of the simplest. Since the moments of inertia of the several sections of the arch vary as the cube of the depth, it will be necessary to make the divisions below the springing line considerably larger than those near the crown and on this account, in order to avoid excessive error, the divisions at the crown should be made comparatively small. The first step is to divide the arch axis into any number of equal parts, which, in the case at hand is 15. Measure the radial depth of the ring at each point of division; determine the length of the arch axis either by dividers or computation, and lay off this length to scale on a horizontal line, as in Fig. 186; divide this line into the same number of equal parts as the half-arch section and at each point of division erect a vertical equal by scale to the moment of inertia at the corresponding point on the arch section. Although the moment of inertia of any arch section, where the section is reinforced with steel, to be exact should be taken as the

sum of the moment of inertia of the concrete section plus n times the moment of inertia of the steel section, for the arches usually designed in sewerage practice it will be sufficiently accurate to consider the moment of inertia of the concrete section alone, neglecting the steel; and since the moment of inertia is proportional to the cube of the depth, the latter quantity may be used instead of the moment of inertia for the length of the vertical line, as specified above. Connect the tops of these verticals by a smooth curve. It may then be assumed that any ordinate to this curve is proportional to the moment of inertia at the corresponding point on the arch ring.

To divide the arch axis into portions of such length that ds/I shall be constant, draw a line ab, at any slope and then a line, bc, at the same slope, and continue the construction by drawing other isosceles triangles as shown, always using the same slope. This divides the rectified arch ring into a number of parts of such length that each part, divided by the moment of inertia at its center, is constant, that is, $ds/I = 2 \tan a$, in which a is the angle between the sides of the isosceles triangle and the vertical.

In Table 149 are given the values used in the above computations for the division of the arch ring.

TABLE 149.—DIVISION OF ARCH RING
Analysis of 15-ft 6-in. × 15-ft. 2-in. Horse-shoe Sewer by Elastic Theory

1 Section number	2 Radial depth of ring "t"	3 t^3	4 Values of ds	1 Section number	2 Radial depth of ring "t"	3 t^3	4 Values of ds
Crown	0.917	0.76	......	11	1.48	3.24	1.12
1	0.92	0.78	0.63	12	1.67	4.66	1.29
2	0.93	0.80	0.63	13	1.92	7.09	1.54
3	0.96	0.88	0.63	14	2.24	11.22	1.96
4	0.99	0.97	0.64	15	2.95	25.65	5.34
5	1.04	1.13	0.66	16	1.90	6.86	4.64
6	1.09	1.30	0.69	17	1.71	5.00	1.92
7	1.15	1.52	0.75	18	1.51	3.44	1.21
8	1.21	1.77	0.80	19	1.30	2.20	0.98
9	1.28	2.10	0.88	20	1.12	1.41	....
10	1.35	2.46	0.98	Invert c.	1.00	1.00	....

It is not important that a point of division shall fall exactly at the end of the horizontal line, but in case it is desired to divide the arch ring into a predetermined number of parts, this can be done by successive approximations. An adjustable angle or protractor will be found of considerable assistance in subdividing the arch ring in this manner.

Conditions.—The sewer section shown is assumed to be subject to an earth fill of 24 ft. above the top of the sewer. The weight of the earth filling is assumed to be 100 lb. per cubic foot, and the angle of repose is taken as 30 deg. It is further assumed that the sewer is to be constructed in rock cut with the side walls and invert resting directly on rock foundation.

Vertical Forces.—The vertical forces acting on the arch section are the weight of the concrete section and the weight of the column of earth above

that section. For purposes of this analysis the weight of the concrete section can usually be omitted, for cases where the vertical load, or the depth of earth fill, above the section is very much larger. Without material error it can also be assumed that the vertical forces act through the center of the axis of the arch for each section. The vertical pressure of the earth above the arch is assumed to be the dead weight of the column of earth, in width equal to the horizontal projection of the extrados section, and in depth equal to the distance from the surface of the ground to the center of the extrados section of the arch. In case the dead weight of the concrete is used, this can be added to the weight of the earth and the resultant pressure applied at the center of the arch axis for each section.

The depth, vertical intensity, horizontal projection of the arch section and total vertical load are tabulated in Table 150, Columns 2 to 5 inclusive. Also the vertical forces are shown graphically in their respective locations in Fig. 186.

Horizontal Forces.—If we assume the angle of repose equal to 30 deg., the intensity of the horizontal earth pressure will be one-third of the intensity of the vertical pressure at any point. The horizontal earth pressure is assumed to act on a width equal to the vertical projection of the extrados section. The values of the horizontal intensity of the earth pressure, the vertical projection of the arch section, and the total horizontal load are given in Table 150, Columns 6, 7 and 8; and the horizontal loads are shown graphically in Fig. 186. The horizontal pressure may be assumed to act at the center of the axis for each section without material error in the final results.

In Columns 9 and 10 are given the successive sums of the vertical and horizontal loads, respectively, at each of the sections. These figures will be used later in the calculation of the moments at the different points.

The co-ordinates of the center points of each section, x and y, referred to the crown as the origin, are shown in Columns 11 and 12, and the values of y^2 in Column 13. Column 14 gives the values of the differences between the successive co-ordinates, as for example $(x_2 - x_1)$, $(x_3 - x_2)$, etc., and Column 15 gives the differences between the y co-ordinates in a similar manner, as for example $(y_2 - y_1)$, $(y_3 - y_2)$, etc.

Bending Moments.—The bending moments (all negative) shown in Column 2 of Table 151 are computed as follows:

$$m_1 = 0$$

$$m_2 = w_1 (x_2 - x_1) + h_1 (y_2 - y_1) = (1610 \times 0.64) + (24 \times 0.03) = 1031$$

$$m_3 = m_2 + \Sigma w_2(x_3 - x_2) + \Sigma h_2(y_3 - y_2) = 1031 + (3197 \times 0.63) + (80 \times 0.12) = 3054$$

$$m_4 = m_3 + \Sigma w_3(x_4 - x_3) + \Sigma h_3(y_4 - y_3) = 3054 + (4791 \times 0.60) + (177 \times 0.13) = 5955$$

$$m_5 = m_4 + \Sigma w_4(x_5 - x_4) + \Sigma h_4(y_5 - y_4) = 5955 + (6419 \times 0.64) + (323 \times 0.19) = 10{,}120$$

$$m_6 = m_5 + \Sigma w_5(x_6 - x_5) + \Sigma h_5(y_6 - y_5) = 10{,}120 + (8035 \times 0.62) + (511 \times 0.24) = 15{,}220$$

$$m_7 = m_6 + \Sigma w_6(x_7 - x_6) + \Sigma h_6(y_7 - y_6) = 15{,}220 + (9669 \times 0.62) + (750 \times 0.33) = 21{,}470$$

Analysis of 15-ft. 6-in. × 15-ft. 2-in. Horse-shoe Sewer by Elastic Theory

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Section number	Depth to center of extrados	Vertical intensity of earth pressure (lb. per sq. ft.)	Horizontal projection of extrados of section, ft.	Total vertical load w, lb.	Horizontal intensity of earth pressure	Vertical projection of extrados of section, ft.	Total horizontal loads h, lb.	Sum of vertical loads Σw	Sum of horizontal loads Σh	Co-ordinates of center of section		y^2	Difference between successive co-ordinates	
										x	y		$x_2 - x_1$, etc.	$y_2 - y_1$, etc.
1	24.01	2,401	0.67	1,610	800	0.03	24	1,610	24	0.32	0.01	0.0001	0.64	0.03
2	24.06	2,406	0.66	1,587	802	0.07	56	3,197	80	0.96	0.04	0.0016	0.63	0.12
3	24.16	2,416	0.66	1,594	805	0.12	97	4,791	177	1.59	0.16	0.0256	0.60	0.13
4	24.30	2,430	0.67	1,628	810	0.18	146	6,419	323	2.19	0.29	0.0841	0.64	0.19
5	24.50	2,450	0.66	1,616	817	0.23	188	8,035	511	2.83	0.48	0.231	0.62	0.24
6	24.75	2,475	0.66	1,634	825	0.29	239	9,669	750	3.45	0.72	0.519	0.62	0.33
7	25.07	2,507	0.69	1,730	836	0.37	309	11,399	1,059	4.07	1.05	1.102	0.67	0.40
8	25.49	2,549	0.72	1,835	850	0.49	416	13,234	1,475	4.74	1.45	2.103	0.67	0.51
9	26.02	2,602	0.72	1,873	867	0.58	503	15,107	1,978	5.41	1.96	3.84	0.67	0.62
10	26.67	2,667	0.72	1,920	889	0.71	631	17,027	2,609	6.08	2.58	6.66	0.69	0.80
11	27.51	2,751	0.73	2,008	917	0.93	853	19,035	3,462	6.77	3.38	11.42	0.65	1.01
12	28.58	2,858	0.70	2,000	953	1.18	1,125	21,035	4,587	7.42	4.39	19.28	0.58	1.29
13	29.94	2,994	0.59	1,766	998	1.56	1,556	22,801	6,143	8.00	5.68	32.26	0.40	1.71
14	31.77	3,177	0.35	1,112	1,059	2.10	2,223	23,913	8,366	8.40	7.39	54.60	0.12	3.66
15	35.49	3,549	0.53	1,881	1,183	5.33	6,305	25,794	14,671	8.52	11.05	122.10	−2.25	3.73
		3,883	0.14	546							40.63	254.23		
16	38.83	−2,680	5.64	−15,050	1,294	1.39	1,800	11,290	16,471	6.27	14.78	218.45	−3.17	0.78
17	......	−2,680	1.92	−5,150	....	....	0	6,140	16,471	3.10	15.56	242.11	−1.53	0.37
18	......	−2,680	1.20	−3,220	....	....	0	2,920	16,471	1.57	15.93	253.76	−1.08	0.20
19	......	−2,680	1.09	−2,920	....	....	0	0	16,471	0.49	16.13	260.18		
											103.03 $= \Sigma y$	1,228.73 $= \Sigma y^2$		

$m_8 = m_7 + \Sigma w_7(x_8 - x_7) + \Sigma h_7(y_8 - y_7) = 21{,}469 +$
$(11{,}399 \times 0.67) + 1059\ (\ \times 0.40) = 29{,}530$

$m_9 = m_8 + \Sigma w_8(x_9 - x_8) + \Sigma h_8\ (y_9 - y_8) = 29{,}530 +$
$(13{,}234 \times 0.67) + (1475 \times 0.51) = 39{,}140$

$m_{10} = m_9 + \Sigma w_9(x_{10} - x_9) + \Sigma h_9(y_{10} - y_9) = 39{,}140 +$
$(15{,}107 \times 0.67) + (1978 \times 0.62) = 50{,}490$

$m_{11} = m_{10} + \Sigma w_{10}(x_{11} - x_{10}) + \Sigma h_{10}(y_{11} - y_{10}) = 50{,}490 +$
$(17{,}027 \times 0.69) + (2609 \times 0.80) = 64{,}303$

$m_{12} = m_{11} + \Sigma w_{11}(x_{12} - x_{11}) + \Sigma h_{11}(y_{12} - y_{11}) = 64{,}330 +$
$(19{,}035 \times 0.65) + (3462 \times 1.01) = 80{,}200$

$m_{13} = m_{12} + \Sigma w_{12}(x_{13} - x_{12}) + \Sigma h_{12}(y_{13} - y_{12}) = 80{,}200 +$
$(21{,}035 \times 0.58) + (4587 \times 1.29) = 98{,}320$

$m_{14} = m_{13} + \Sigma w_{13}(x_{14} - x_{13}) + \Sigma h_{13}(y_{14} - y_{13}) = 98{,}320 +$
$(22{,}801 \times 0.40) + (6143 \times 1.71) = 117{,}940$

$m_{15} = m_{14} + \Sigma w_{14}(x_{15} - x_{14}) + \Sigma h_{14}(y_{15} - y_{14}) = 117{,}940 +$
$(23{,}913 \times 0.12) + (8366 \times 3.66) = 151{,}430$

$m_{16} = m_{15} + \Sigma w_{15}(x_{16} - x_{15}) + \Sigma h_{15}(y_{16} - y_{15}) = 151{,}430 +$
$(25{,}794 \times -2.25) + (14{,}671 \times 3.73) = 148{,}120$

$m_{17} = m_{16} + \Sigma w_{16}(x_{17} - x_{16}) + \Sigma h_{16}(y_{17} - y_{16}) = 148{,}120 +$
$(11{,}290 \times -3.17) + (16{,}471 \times 0.78) = 125{,}170$

$m_{18} = m_{17} + \Sigma w_{17}(x_{18} - x_{17}) + \Sigma h_{17}(y_{18} - y_{17}) = 125{,}170 +$
$(6140 \times -1.53) + (16{,}471 \times 0.37) = 121{,}875$

$m_{19} = m_{18} + \Sigma w_{18}(x_{19} - x_{18}) + \Sigma h_{18}(y_{19} - y_{18}) = 121{,}875 +$
$(2920 \times -1.08) + (16{,}471 \times 0.20) = 122{,}019$

TABLE 151.—BENDING MOMENTS, THRUSTS AND SHEARS. CASE I

Analysis of 15-ft. 6-in. × 15-ft. 2-in. Horseshoe Sewer by Elastic Theory

1 Section number	2 Bending moments, m	3 my	4 H_oy	5 Total bending moments, ft.-lb.	6 Thrusts, lb.	7 Eccentric distances, ft.	8 Shears, lb.
1	0	....	146	+6,584	14,550	+0.45	1,100
2	−1,031	−41	583	+5,990	14,720	+0.41	1,550
3	−3,054	−489	2,330	+5,714	14,990	+0.38	2,000
4	−5,955	−1,726	4,223	+4,706	15,400	+0.31	2,500
5	−10,120	−4,860	6,990	+3,308	15,900	+0.21	2,900
6	−15,220	−10,960	10,490	+1,708	16,530	+0.10	3,200
7	−21,470	−22,540	15,290	+258	17,330	+0.01	3,500
8	−29,530	−42,820	21,110	−1,982	18,210	−0.11	3,600
9	−39,140	−76,710	28,540	−4,162	19,330	−0.22	3,500
10	−50,490	−130,300	37,570	−6,482	20,500	−0.32	3,200
11	−64,330	−217,400	49,220	−8,672	21,860	−0.40	2,600
12	−80,200	−352,100	63,930	−9,832	23,210	−0.42	1,300
13	−98,320	−558,500	82,710	−9,172	24,300	−0.38	550
14	−117,940	−871,600	107,610	−3,892	24,500	−0.16	3,200
15	−151,430	−1,673,000	160,910	+15,918	25,800	+0.62	110
	−688,230 = Σm	−3,963,046 = Σmy					

Computations for m_{16} to m_{19} are for Case II.

The summations $w_1 + w_2 + w_3$ and $h_1 + h_2 + h_3$, etc., are taken from Columns 9 and 10, Table 150. The difference between the x and y coordinates, as $(x_2 - x_1)$ and $(y_2 - y_1)$, are taken from Columns 14 and 15, respectively, of the same table.

Forces at Crown.—The next step in the analysis is to find the crown thrust, which can be obtained from the equation previously given. The values of Σm, Σmy, Σy and Σy^2 are given in Columns 2 and 3 of Table 151 and Columns 12 and 13 of Table 150.

$$H_o = \frac{n\Sigma my - \Sigma m\Sigma y}{(\Sigma y)^2 - n\Sigma y^2} = \frac{15(-3,963,046) - (-688,230 \times 40.63)}{40.63^2 - 15 \times 254.23}$$

$$H_o = 14,562 \text{ pounds}$$

In the above equation n = the number of divisions in the half arch section.

The bending moments at the crown can also be obtained by the equations already given, as follows:

$$M_o = -\frac{\Sigma m + H_o\Sigma_y}{n} = -\frac{-688,230 + 14,562 \times 40.63}{15}$$

$$M_o = +6438 \text{ ft.-lb.}$$

The values of the crown thrust H_o multiplied by the values of y, are computed and tabulated in Column 4, Table 151.

From the data thus obtained the total bending moment for each section is computed from the formula given in a previous paragraph, as follows:

$$M = m + M_o + H_oy$$

$$M_1 = m_1 + M_o + H_oy_1 = 0 + 6438 + 146 = +6584$$

$$M_2 = m_2 + M_o + H_oy_2 = -1031 + 6438 + 583 = +5990$$

The results are recorded in Column 5 Table 151.

Force Diagram.—The value of the thrust and shear at any point can be obtained from the force diagram by graphical methods. As a rule, the shear in sewer arches can be neglected. The value of H_o = 14,562 lb., the crown thrust, is first laid off to scale on a horizontal line, as shown in Fig. 186. At the left end of this line, lay off to scale the vertical force w_1, vertically downward, and at its lower extremity lay off the horizontal force h_1 to scale horizontally to the right. A line drawn to connect the right hand end of h_1 with the upper end of w_1 is equal in amount, by scale and direction, to the resultant external force P_1 acting on the first section of the arch. A line or "ray" drawn from the right extremity of h_1 to the right extremity of the horizontal line H_o or origin, is equal in amount, by scale and direction, to the resultant pressure, R_1, between sections 1 and 2 of the arch. At the right extremity of h_1, lay off w_2 vertically downward and then h_2 horizontally to the right and so on for each successive vertical and horizontal load acting on the arch. The broken line thus formed is called the "load line." The resultant external force acting on each section of the arch can be obtained as above, and also the resultant pressure acting between the sections of the arch. The

normal or true thrust is obtained by resolving the force R parallel and normal to the arch axis at the point in question. These are shown by the dotted lines on the force diagram, Fig. 186.

Equilibrium Polygon.—The equilibrium polygon, or diagram showing the line of pressure on the arch, is drawn by the aid of the force diagram. The crown thrust acts for a symmetrically loaded arch in a horizontal direction, and the point of application is at a distance above the axis of the arch at the crown equal to $M_o/H_o = e$, the eccentric distance, if M_o is plus, and below the axis by the same amount if M_o is minus. For the example at hand

$$\frac{M_o}{H_o} = \frac{+6438}{14,562} = +0.442 \text{ ft.}$$

This distance is then laid off vertically above the arch axis at the crown, and the resultant crown thrust is drawn through this point to its intersection with the resultant external force acting on the first section of the arch. From this point of intersection draw a line parallel to R_1, taken from the force diagram, and prolong it to an intersection with the oblique force acting on section 2 of the arch. In a similar manner continue by taking the "rays" from the force diagram and prolong each to its intersection with the next oblique force acting on the arch.

The intersection with the first joint of a line parallel to R_1 from the force diagram, gives the center of pressure on that joint, and the intersection of R_2 with the second joint gives the center of pressure for that joint, and so on for the other joints. The broken line thus obtained, passing down through the arch section, is the "line of resistance," or the "line of thrust" for the arch. The amount of each thrust, that is, the true thrust normal to the section (for practical purposes, the total resultant pressure, may often be taken as the normal thrust without serious error) should be scaled from the force diagram and the amounts recorded in Column 5 of Table 151.

The eccentric distance, e, is found by dividing the total bending moment by the thrust, and is recorded in Column 6 of Table 151.

For positive moments, and therefore positive values of e, the line of thrust lies above the arch axis. The amount of the eccentricity is shown graphically on the diagram of the arch by the distance from the arch axis to the point of application of the thrust, which is the intersection of the line of pressure with the plane of the section. After the line of resistance or equilibrium polygon has been drawn, the computed values of the eccentricity can be checked by scaling the values on the equilibrium polygon. While it is not necessary to draw the "line of resistance" or "equilibrium polygon" in order to obtain the fiber stresses, it is usually well to do so in order to check the algebraic work.

It should be borne in mind that the equilibrium polygon does not give the true line of resistance. As the number of subdivisions of the axis are increased the equilibrium polygon approaches the true line of resistance, which is a curve. The exact values of the eccentricity e are the distances between the arch axis and the curve of resistance, measured on a line perpendicular to the tangent to the axis at the center points in question. For practical purposes the equilibrium polygon is sufficiently near the true line of resistance.

It will be noted that in the analysis under Case II the difference is more noticeable near the base of the side wall and in the invert.

Analysis of Case II.—In the preceding analysis, the invert of the section was considered as separated from the side wall, but in this analysis, under Case II, the entire structure will be analyzed. The same assumption, as to vertical and horizontal forces acting on the arch and side wall are made and in addition it is assumed that there are vertical forces acting upward on the invert equal in amount to the total downward vertical forces, and uniformly distributed over the invert. See Fig. 186. The upward vertical force acting on block 16 is combined with the vertical (downward) and horizontal components of the earth pressure acting on the left side of the block producing the oblique resultant force as shown.

Division of Axis to make ds/I Constant.—The chief disadvantage of this method as applied to Case II lies in the necessity of dividing the axis according to a prescribed ratio. This usually requires careful manipulation and repeated trials to subdivide the side wall and invert in order to obtain suitable divisions. It can be done as Fig. 186 shows, and in the example at hand no great difficulty was experienced. Blocks 15 and 16 are, however, somewhat larger than is desirable for sections where large thrusts, bending moments and shears occur.

The method of dividing the axis is the same as described under Case I.

Computations.—The remainder of the computations are made in the same manner as for Case I. New values of H_o, M_o and e are computed, using the summations from division 1 to 19 inclusive instead of from 1 to 15 inclusive as in Case I. For convenience the values for points 16 to 19 have been included with the others in Table 150. The computations of the bending moments for joints 16 to 19 were given with those arising under the assumptions of Case I.

New values of H_o, M_o and e are found from the formulas as before, using the summations from divisions 1 to 19 inclusive, as follows:

$$H_o = \frac{n\Sigma my - \Sigma m\Sigma y}{(\Sigma y)^2 - n\Sigma y^2} = \frac{19(-12{,}009{,}546) - (-1{,}205{,}414 \times 103.03)}{103.03^2 - (19 \times 1228.73)}$$

$$H_o = +8170 \text{ lb.}$$

$$M_o = -\frac{\Sigma m + H_o\Sigma y}{n} = -\left[\frac{-1{,}205{,}414 + (8170 \times 103.03)}{19}\right]$$

$$M_o = +19{,}140 \text{ ft.-lb}$$

$$e = \frac{M_o}{H_o} = \frac{+19{,}140}{+8170} = +2.34$$

With the above values a new force diagram is drawn (Fig. 186) and a new equilibrium polygon in the same manner as for Case I.

The computations for the bending moments, thrusts, shears and eccentric distances are given in Table 152.

A comparison of the two equilibrium polygons or lines of resistance for the two cases shows plainly how the bending moments are greatly increased by the addition of the invert as part of the elastic structure.

Having given the amount and point of application of the normal thrust on each joint or section of the arch, the resulting fiber stresses can be readily

normal or true thrust is obtained by resolving the force R parallel and normal to the arch axis at the point in question. These are shown by the dotted lines on the force diagram, Fig. 186.

Equilibrium Polygon.—The equilibrium polygon, or diagram showing the line of pressure on the arch, is drawn by the aid of the force diagram. The crown thrust acts for a symmetrically loaded arch in a horizontal direction, and the point of application is at a distance above the axis of the arch at the crown equal to $M_o/H_o = e$, the eccentric distance, if M_o is plus, and below the axis by the same amount if M_o is minus. For the example at hand

$$\frac{M_o}{H_o} = \frac{+6438}{14,562} = +0.442 \text{ ft.}$$

This distance is then laid off vertically above the arch axis at the crown, and the resultant crown thrust is drawn through this point to its intersection with the resultant external force acting on the first section of the arch. From this point of intersection draw a line parallel to R_1, taken from the force diagram, and prolong it to an intersection with the oblique force acting on section 2 of the arch. In a similar manner continue by taking the "rays" from the force diagram and prolong each to its intersection with the next oblique force acting on the arch.

The intersection with the first joint of a line parallel to R_1 from the force diagram, gives the center of pressure on that joint, and the intersection of R_2 with the second joint gives the center of pressure for that joint, and so on for the other joints. The broken line thus obtained, passing down through the arch section, is the "line of resistance," or the "line of thrust" for the arch. The amount of each thrust, that is, the true thrust normal to the section (for practical purposes, the total resultant pressure, may often be taken as the normal thrust without serious error) should be scaled from the force diagram and the amounts recorded in Column 5 of Table 151.

The eccentric distance, e, is found by dividing the total bending moment by the thrust, and is recorded in Column 6 of Table 151.

For positive moments, and therefore positive values of e, the line of thrust lies above the arch axis. The amount of the eccentricity is shown graphically on the diagram of the arch by the distance from the arch axis to the point of application of the thrust, which is the intersection of the line of pressure with the plane of the section. After the line of resistance or equilibrium polygon has been drawn, the computed values of the eccentricity can be checked by scaling the values on the equilibrium polygon. While it is not necessary to draw the "line of resistance" or "equilibrium polygon" in order to obtain the fiber stresses, it is usually well to do so in order to check the algebraic work.

It should be borne in mind that the equilibrium polygon does not give the true line of resistance. As the number of subdivisions of the axis are increased the equilibrium polygon approaches the true line of resistance, which is a curve. The exact values of the eccentricity e are the distances between the arch axis and the curve of resistance, measured on a line perpendicular to the tangent to the axis at the center points in question. For practical purposes the equilibrium polygon is sufficiently near the true line of resistance.

material taken symmetrically on both sides of the center of the invert is acted upon by direct stresses and bending moments (see Fig. 187). The study section, instead of acting as a cantilever beam, as in Fig. 185, acts like an elastic ring, Fig. 187. Vertical and horizontal earth pressures are assumed to act on the semi-circular arch and side walls and the upward pressure on the bottom is assumed to be uniformly distributed over the bottom and equal in total amount to the sum of the downward vertical forces. Such a distribution of the upward forces seems to be a reasonable assumption if the sewer is constructed on yielding or compressible soil, and at any rate it imposes more severe conditions than the assumption that the upward forces are distributed with greater intensity near the side walls.

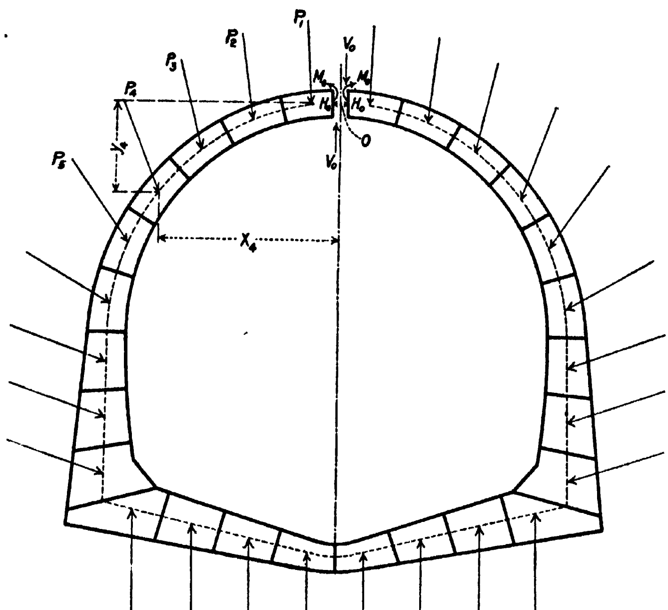


FIG. 187.

Case II.—In this case the same assumptions are made as to vertical and horizontal earth loads on the semi-circular arch and side walls, but the invert is considered as separated from the side, possibly by joints at the junction of invert and side. The invert serves only as a tie or strut to space the walls and carries none of the vertical reaction. In this case the elastic deformation of only the semi-circular arch and side wall is considered. This method is the same in theory as the analysis for the elastic arch previously given, but differs in method because in the latter, the arch ring is divided so as to make

ds/I constant, while the method described in the following pages makes no such division.

There should be but little difference in the lines of pressure from the method given by Turneaure and Maurer and that described in the following paragraphs. Some differences occur due to the approximations used which might be eliminated if greater precision was justified. The results are sufficiently close, bearing in mind the uncertainties of loading, earth pressures, etc.

Case I.—If the sewer section shown in Fig. 187 is cut at the crown by a vertical plane, the structure may be considered as a curved beam acted upon by the known external loads and the unknown forces, H_o, V_o and M_o. If these three unknown forces are determined the resultant force acting at any section may be found, either analytically or graphically.

Let H_o = thrust at the crown (Fig. 187).
V_o = shear at the crown.
M_o = bending moment at the crown.
m = bending moment at any section center due to the external loads on one side of the section, the ring being considered as a curved beam; negative in sign for left-hand half of arch.
N, V, and M = thrust, shear, and total moment at any section center.
ds = length of a division of the arch ring measured along the arch axis.
I = moment of inertia of any section, determined at the center.
w, h, P = the vertical, horizontal and resultant external forces, respectively acting on the sewer section.
R = the resultant pressure at any section.
x, y = co-ordinates of any point on the axis of the sewer section referred to the crown as origin. All considered as positive in sign.
m_y = moment at any section due to 1 lb. acting vertically at O.
m_x = moment at any section due to 1 lb. acting horizontally at O, the crown.
d_{1y} = vertical deflection of O due to 1 lb. acting vertically at O.
d_{1x} = horizontal deflection of O due to 1 lb. acting vertically at O.
d_{1a} = angular change of face at O due to 1 lb. acting vertically at O.
d_{2y} = vertical deflection of O due to 1 lb. acting horizontally at O.
d_{2x} = horizontal deflection of O due to 1 lb. acting horizontally at O.
d_{2a} = angular change of face at O due to 1 lb. acting horizontally at O.
d_{3y} = vertical deflection of O due to 1 in.-lb. bending moment at O.
d_{3x} = horizontal deflection of O due to 1 in.-lb. bending moment at O.

d_{3a} = angular change of face at O due to 1 in.-lb. bending moment at O.
Δ_y = vertical deflection of O due to external forces.
Δ_x = horizontal deflection of O due to external forces.
Δ_a = angular change of face at O due to external forces.

Assume deflections to the right and upward as having a positive sign, and deflections in the opposite direction as negative. Assume that revolutions or angular changes of face at O in a clockwise direction have a positive sign.

Equations.—From the fact that the vertical, horizontal and angular deflection of the right and left faces of the crown joint must be identical, the three following equations can be derived.

$$-\Delta_y + V_o d_{1y} + H_o d_{2y} + M_o d_{3y} = -\Delta_y - V_o d_{1y} + H_o d_{2y} + M_o d_{3y}$$
$$\Delta_x - H_o d_{2x} - M_o d_{3x} = -\Delta_x + H_o d_{2x} + M_o d_{3x}$$
$$\Delta_a - H_o d_{2a} - M_o d_{3a} = -\Delta_a + H_o d_{2a} + M_o d_{3a}$$

From the first equation we obtain $V_o = 0$ and on that account it has been omitted in the second and third equations.

From the second and third equations the following values can be obtained:

$$H_o = \frac{\Delta_a d_{3x} - \Delta_x d_{3a}}{d_{2a} d_{3x} - d_{2x} d_{3a}}$$

$$M_o = \frac{\Delta_a d_{2x} - \Delta_x d_{2a}}{d_{3a} d_{2x} - d_{3x} d_{2a}}$$

Considering only the deflections needed for the solution of equations for H_o and M_o, their values may be computed from the formulas:

$$\Delta_a = \Sigma\, m \frac{ds}{EI}, \quad \text{taken as} = \Sigma\, m \frac{ds}{t^3}$$

$$\Delta_x = \Sigma\, m m_x \frac{ds}{EI} \text{ taken as} = \Sigma\, m m_x \frac{ds}{t^3}$$

$$d_{2a} = d_{3x} = \Sigma\, m_x \frac{ds}{EI}, \quad \text{taken as} = \Sigma\, m_x \frac{ds}{t^3}$$

$$d_{3a} = \Sigma \frac{ds}{EI}, \quad \text{taken as} = \Sigma \frac{ds}{t^3}$$

$$d_{2x} = \Sigma\, m_x^2 \frac{ds}{EI}, \quad \text{taken as } \Sigma\, m_x^2 \frac{ds}{t^3}$$

where t = thickness of masonry ring at the center of any section.

$$M = m + M_o + H_o y$$

In the above formulas for H_o and M_o each expression represents the summation of the values indicated for the several divisions of the axis under consideration.

In the computations the factor 1/12 in the moment of inertia, I, is omitted as being common to all terms; also the coefficient of elasticity, E, as shown in the above equations is omitted as it is constant throughout.

Division of Arch Section.—The first step in the analysis is to draw the half

sewer section to some convenient scale of suitable size to allow the scaling of various dimensions and forces without causing too great an error. This section is shown in Fig. 188. The center line of the section, shown by the dotted line, is divided into a number of divisions which, for convenience, may be approximately equal, although this is not necessary. The section shown has been divided into 13 divisions. By this method it is not necessary to subdivide the arch axis into divisions so that ds/I shall be constant, as in the method previously described. This has the advantage of allowing the side wall and invert to be divided into sections convenient for computation, and especially at the junction between the side wall and invert it makes it possible to determine the bending moment with greater accuracy.

Computations.—The radial thickness of the masonry ring at the center of each section is then scaled from the drawing and recorded in Column 2 of Table 153. The cube of the thickness for each section is recorded in Column 3 and the length of each section measured along the axis of the arch is recorded in Column 4. Column 5 gives the values, for each division, of ds/t^3, which is equivalent to d_{3a}.

In Columns 6 and 7 of Table 153 are given the co-ordinates of the center point of each division, the x co-ordinate being measured horizontally from the crown and the y co-ordinate being measured vertically from the center of the arch division to the center of the crown joint.

TABLE 153.—COMPUTATIONS OF EXTERNAL FORCES AND MOMENTS

Analysis of 15 ft. 6 in. × 15 ft. 2 in. Horse-shoe Sewer by Method for Indeterminate Structures

1	2	3	4	5	6	7	8	9	10	11
Section No.	Thickness of ring at center of section t, ft.	t^3, ft.	ds, ft.	$\frac{ds}{t^3}$, (d_{3a}), ft.	Co-ordinates of center of section		$m_x = y \times 1$ lb. ft. lb.	$(m_x)^2$	$m_x^2 \frac{ds}{t^3}$ (d_{2x})	$m_x \frac{ds}{t^3}$ $d_{2a} = d_{3x}$
					x, ft.	y, ft.				
1	0.93	0.8043	2.21	2.748	1.09	0.08	0.08	0.0064	0.018	0.220
2	0.95	0.8573	2.21	2.578	3.18	0.62	0.62	0.3844	0.991	1.598
3	1.03	1.0927	2.21	2.023	5.09	1.70	1.70	2.8900	5.847	3.439
4	1.13	1.4429	2.21	1.532	6.68	3.20	3.20	10.2410	15.688	4.902
5	1.23	1.8608	2.21	1.187	7.76	5.08	5.08	25.8030	30.630	6.030
6	1.36	2.5154	2.21	0.878	8.36	7.17	7.17	51.4090	45.140	6.204
7	1.57	3.8699	2.02	0.522	8.46	9.28	9.28	86.1180	44.950	4.844
8	1.98	7.7624	2.02	0.260	8.48	11.30	11.30	127.6900	33.200	2.938
9	2.56	16.7772	2.02	0.120	8.48	13.32	13.32	177.4220	21.290	1.599
				11.848					197.754	31.864
10	1.94	7.3014	2.18	0 299	7.43	14.59	14.59	212.8680	63.650	4.363
11	1.62	4.2515	2.18	0.513	5.32	15.07	15.07	227.1050	116.500	7.732
12	1.35	2.4604	2.18	0.886	3.20	15.55	15.55	241.8020	214.250	13.777
13	1.05	1.1576	2.18	1.883	1.07	16.03	16.03	256.9610	483.860	30.185
				15.429					1,076.014	87.921

In Column 8 are given values of m_x, the moment at each of the sections, due to a force of 1 lb. acting horizontally at the crown. This is equivalent to 1 lb. multiplied by the y co-ordinate at each center point. Column 9 gives

the values of m_x^2 and Column 10 the values of $m_x^2 \frac{ds}{t^3}$, which will be required later for the values of d_{2x}. Column 11 gives the values of $m_x \frac{ds}{t^3}$, which will give the values of d_{2a} and its equal, d_{3x}.

External Forces.—Table 154 shows the computations for the external forces which are made in the same manner as the computations for the vertical and horizontal forces described under the analysis of the elastic arch (see Table 150). The sewer section shown in Fig. 188 is the same section that was used for the analysis of the elastic arch shown in Fig. 186. The depth of earth fill over the extrados of the section at the crown is assumed to be 24 ft., the unit weight of earth being 100 lb. per cubic foot and the angle of repose, 30 deg.

As the method of computing the data given in Table 154, has already been carefully explained it will not be repeated here.

Computation of Partial Bending Moments.—In Table 154, Column 11, are given the values of the differences of the co-ordinates, as for example, $(x_2 - x_1)$, $(x_3 - x_2)$, etc. Column 12 gives the differences of the y co-ordinates. Column 13 shows the bending moments (all negative) of the external loads computed for each section as follows:

$$m_1 = 0$$
$$m_2 = w_1(x_2 - x_1) + h_1(y_2 - y_1) = (5{,}475 \times 2.09) + (224 \times 0.54) = 11{,}563$$
$$m_3 = m_2 + \Sigma w_2(x_3 - x_2) + \Sigma h_2(y_3 - y_2) = 11{,}563 + (10{,}725 \times 1.91) + (912 \times 1.08) = 33{,}033$$
$$m_4 = m_3 + \Sigma w_3(x_4 - x_3) + \Sigma h_3(y_4 - y_3) = 33{,}033 + (15{,}465 \times 1.59) + (2{,}067 \times 1.50) = 60{,}723$$
$$m_5 = m_4 + \Sigma w_4(x_5 - x_4) + \Sigma h_4(y_5 - y_4) = 60{,}723 + (19{,}445 \times 1.08) + (3{,}682 \times 1.88) = 88{,}645$$
$$m_6 = m_5 + \Sigma w_5(x_6 - x_5) + \Sigma h_5(y_6 - y_5) = 88{,}645 + (22{,}335 \times 0.60) + (5{,}752 \times 2.09) = 114{,}060$$
$$m_7 = m_6 + \Sigma w_6(x_7 - x_6) + \Sigma h_6(y_7 - y_6) = 114{,}060 + (23{,}725 \times 0.10) + (8{,}192 \times 2.11) = 133{,}717$$
$$m_8 = m_7 + \Sigma w_7(x_8 - x_7) + \Sigma h_7(y_8 - y_7) = 133{,}717 + (24{,}535 \times 0.02) + (10{,}552 \times 2.02) = 155{,}527$$
$$m_9 = m_8 + \Sigma w_8(x_9 - x_8) + \Sigma h_8(y_9 - y_8) = 155{,}527 + (25{,}392 \times 0) + (13{,}172 \times 2.02) = 182{,}137$$
$$m_{10} = m_9 + \Sigma w_9(x_{10} - x_9) + \Sigma h_9(y_{10} - y_9) = 182{,}137 + (26{,}447 \times -1.05) + (16{,}512 \times 1.27) = 175{,}339$$
$$m_{11} = m_{10} + \Sigma w_{10}(x_{11} - x_{10}) + \Sigma h_{10}(y_{11} - y_{10}) = 175{,}339 + (17{,}747 \times -2.11) + (16{,}512 \times 0.48) = 145{,}815$$
$$m_{12} = m_{11} + \Sigma w_{11}(x_{12} - x_{11}) + \Sigma h_{11}(y_{12} - y_{11}) = 145{,}815 + (11{,}987 \times -2.12) + (16{,}512 \times 0.48) = 128{,}331$$
$$m_{13} = m_{12} + \Sigma w_{12}(x_{13} - x_{12}) + \Sigma h_{12}(y_{13} - y_{12}) = 128{,}331 + (6{,}197 \times -2.13) + (16{,}512 \times 0.48) = 123.057$$

From the values of the moments given in Column 13, Table 154 and the data in Columns 5 and 11, Table 153, the values of Δ_a and Δ_x shown in Columns 14 and 15, of Table 154, can be computed.

TABLE 154.—COMPUTATIONS OF EXTERNAL FORCES AND MOMENTS

Analysis of 15 ft. 6 in. × 15 ft. 2 in. Horseshoe Sewer by Method for Indeterminate Structures

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Section number	Depth to center of extrados, ft.	Vertical intensity of earth pressure. lb. per sq. ft.	Horizontal projection of extrados section, ft.	Total vertical load "w" lb.	Horizontal intensity of earth pressure, lb. per sq. ft.	Vertical projection of extrados section, ft.	Total horizontal load h, lb.	Sum of vertical loads Σw, lb.	Sum of horizontal loads Σh, lb.	Difference between successive co-ordinates: $x_2 - x_1$, etc., ft.	$y_2 - y_1$, etc., ft	Bending moments m, ft. lb.	$m\frac{ds}{t^3}$ (Δ_a)	$mm\ x\ \frac{ds}{t^3}$ (Δ_x)
1	24.0	2,400	2.28	5,475	800	0.28	224	5,475	224	2.09	0.54	0	....	....
2	24.5	2,450	2.14	5,250	817	0.84	688	10,725	912	1.91	1.08	11,563	30,045	18,623
3	25.5	2,550	1.86	4,740	850	1.36	1,155	15,465	2,067	1.59	1.50	33,033	66,830	113,600
4	26.9	2,690	1.48	3,980	897	1.80	1,615	19,445	3,682	1.08	1.88	60,723	93,205	297,640
5	29.5	2,950	0.98	2,890	983	2.10	2,070	22,335	5,752	0.60	2.09	88,645	105,230	534,500
6	31.6	3,160	0.44	1,390	1,053	2.32	2,440	23,725	8,192	0.10	2.11	114,060	100,050	717,960
7	33.7	3,370	0.24	810	1,123	2.10	2,360	24,535	10,552	0.02	2.02	133,717	69,800	647,700
8	35.7	3,570	0.24	857	1,190	2.20	2,620	25,392	13,172	0.00	2.02	155,527	40,440	457,000
9	37.7	3,770	0.28	1,055	1,257	2.66	3,340	26,447	16,512	−1.05	1.27	182,137	21,855	291,230
													527,455	3,078,253
10	......	....	3.26	−8,700	....	....	......	17,747	16,512	−2.11	0.48	175,339	52,430	765,000
11	......	....	2.16	−5,760	....	....	......	11,987	16,512	−2.12	0.48	145,815	74,805	1,127,500
12	......	....	2.17	−5,790	....	....	......	6,197	16,512	−2.13	0.48	128,331	113,700	1,768,000
13	......	....	2.31	−6,170	....	....	......	0	....	....	....	123,057	231,750	3,714,800
													1,000,140	10,453,553

Crown Thrust.—From the data at hand it is now possible to compute the value of H_o, the crown thrust, from the formula previously given,

$$H_o = \frac{\Delta_a d_{1x} - \Delta_x d_{1a}}{d_{1a}d_{1x} - d_{1x}d_{1a}} = \frac{(1{,}000{,}140 \times 87.921) - (10{,}453{,}553 \times 15.429)}{(87.921 \times 87.921) - (1076.014 \times 15.429)}$$

$$H_o = +\,8{,}260$$

Moment at the Crown.—The moment at the crown, M_o, can also be computed from the formula already given, as follows:

$$M_o = \frac{\Delta_a d_{2x} - \Delta_x d_{2a}}{d_{1a}d_{2x} - d_{1x}d_{2a}} = \frac{(1{,}000{,}140 \times 1076.014) - (10{,}453{,}553 \times 87.921)}{(15.429 \times 1076.014) - (87.921 \times 87.921)}$$

$$M_o = +\,17{,}700.$$

Eccentricity at the Crown.—From the values just computed the eccentricity at the crown, e, can be obtained as follows:

$$e = M_o/H_o = 17{,}700/8{,}260 = 2.14.$$

If M_o is positive in sign, the value of e will also be positive in sign and the distance, e that is, from the arch axis to the point of application of the crown thrust, should be measured vertically upward from the arch axis. If, on the other hand, the value of M_o had a negative sign, the corresponding value of the eccentric distance, e, would have a negative sign and the eccentric distance should in that case be measured vertically downward from the arch axis at the crown.

The total bending moments, M, at each center point can now be computed from the formula

$$M = m + M_o + H_o y$$

The fact should be borne in mind that for the left half of the structure (the half considered in this analysis) the values of the bending moments, m, all have a negative sign.

TABLE 155.—BENDING MOMENTS, THRUSTS AND SHEARS—CASE I

Analysis of 15 ft. 6 in. × 15 ft. 2 in. Horseshoe Sewer by Method for Indeterminate Structures

1 Section	2 $H_o y$ ft. lb.	3 Total bending moments, M, ft. lb.	4 Thrusts, N, lb.	5 Eccentric distances, e, ft.	6 Shears, V, lb.
Crown		17,700			
1	660	18,360	8,670	+2.11	4,400
2	5,120	11,257	10,750	+1.04	7,200
3	14,050	− 1,283	14,200	−0.10	8,600
4	26,410	−16,613	18,000	−0.93	8,500
5	41,990	−28,955	21,400	−1.36	6,750
6	59,200	−37,160	23,400	−1.59	3,700
7	76,650	−39,367	24,600	−1.60	2,300
8	93,400	−44,427	25,400	−1.75	5,000
9	110,000	−54,437	26,450	−2.06	8,250
10	120,500	−37,139	12,100	−3.08	15,500
11	124,500	−3.615	10,800	−0.34	9,850
12	128,500	+17,869	9,480	+1.87	4,200
13	132,400	+27,043	8,100	+3.33	1,800

Column 3 Table 155 gives the values thus obtained.

Force Diagram.—From the data at hand the force diagram can be constructed in the same manner as described under the analysis of the elastic arch. The stress at the crown, $H_o = +8260$, is laid off on a horizontal line, as shown in Fig. 188, and the load line of the external forces constructed, from which the values of the thrusts can be obtained The values of the thrusts are entered in Column 4 of Table 155.

Equilibrium Polygon.—The equilibrium polygon can now be constructed in the same manner as described under the analysis of the elastic arch. The crown thrust is located at a distance from the axis equal to the eccentric distance already found, or + 2.14 ft. This distance is laid off vertically upward from the arch axis at the crown and the crown thrust is extended to its intersection with the first oblique external force acting on section 1 of the arch. The remainder of the polygon can be constructed in accordance with the analysis of the elastic arch already described. Referring to Fig. 188 it will be noted that between sections 9 and 10 the equilibrium polygon doubles back on itself. This is of no particular importance, but is liable to be confusing unless special care is taken in scaling the eccentric distances from the axis to the correct lines of resistance. If the line of the equilibrium polygon from one external force to the next is followed in logical order, there should be no trouble. A different arrangement of the divisions or a combination of the external forces acting on sections 9 and 10 would practically eliminate this peculiarity without changing the location of the lines in the remaining sections.

It is interesting to note that the line of resistance lies outside of the masonry section for almost its entire distance, and at the invert it is considerably below the section.

As previously stated, it is not necessary to draw the equilibrium polygon in order to obtain the stresses at the various points in the section, but it is usually advisable to do so in order to obtain the advantage of checking the algebraic work by scaling the eccentric distances from the diagram, for comparison with those computed and recorded in Column 5 of Table 155. If these distances, or any one or more of them, do not check with reasonable accuracy, the computation should be inspected for possible errors.

Computations for Case II.—In the figures used for Case I the summations included sections 1 to 13, inclusive, while for Case II the summations should include only sections 1 to 9 inclusive. For Case II new values of H_o, the crown thrust, and M_o, the moment at the crown, are computed, using the figures for sections 1 to 9, inclusive. These new computations are given below.

$$H_o = \frac{\Delta_a d_{3x} - \Delta_x d_{3a}}{d_{2a}d_{3x} - d_{2x}d_{3a}} = \frac{(527{,}455 \times 31.864) - (3{,}078{,}253 \times 11.848)}{(31.864 \times 31.864) - (197.754 \times 11.848)}$$

$$H_o = +\ 14{,}810.$$

$$M_o = \frac{\Delta_a d_{2x} - \Delta_x d_{2a}}{d_{3a}d_{2x} - d_{3x}d_{2a}} = \frac{(527{,}455 \times 197.754) - (3{,}078{,}253 \times 31.864)}{(11.848 \times 197.754) - (31.864 \times 31.864)}$$

$$M_o = +\ 4{,}680.$$

The new eccentric distance is obtained as before,

$$e = \frac{M_o}{H_o} = \frac{+\ 4{,}680}{14{,}810} = 0.316 \text{ ft.}$$

TABLE 156.—BENDING MOMENTS, THRUSTS AND SHEARS—CASE II

Analysis of 15 ft. 6 in. × 15 ft. 2 in. Horse-shoe Sewer by Method for Indeterminate Structures

1 Section	2 H_0y, ft. lb.	3 Total bending moments, ft lb.	4 Thrusts, N, lb.	5 Eccentric distances, e, ft.	6 Shears, V, lb.
Crown		+ 4,680	14,810	+0.32	
1	1,180	+ 5,860	15,120	+0.39	3,500
2	9,190	+ 2,307	16,830	+0.14	4,700
3	25,190	− 3,163	19,400	−0.16	4,650
4	47,400	− 8,643	22,000	−0.39	3,300
5	75,300	− 8,665	24,000	−0.36	750
6	106,200	− 3,180	24,400	−0.13	2,800
7	137,400	+ 8,363	24,535	+0.34	4,200
8	167,400	+16,553	25,390	+0.65	1,500
9	197,300	+19,843	26,450	+0.75	1,700

A new force diagram can now be constructed, using the same load line as before, see Fig. 188. The horizontal distance H_o, or thrust at the crown, will be different and on that account the amount and direction of the rays will be different.

From the new force diagram another equilibrium polygon can be laid out on the masonry section, as shown in Fig. 188, the dash line being the line

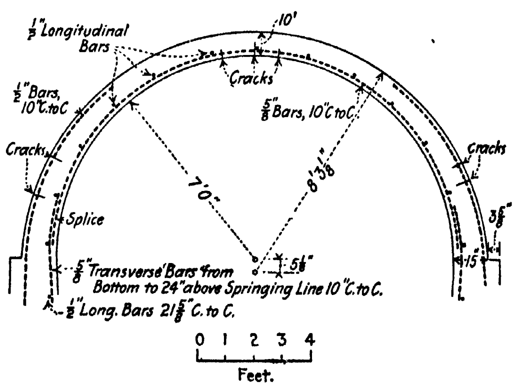


FIG. 189.—Cracks in sewer arch caused by excessive loading.

of resistance or equilibrium polygon for Case II, while the dot and dash line is the equilibrium polygon for Case I.

It is of interest to compare the two lines of resistance, as showing the difference in the internal stresses set up in the masonry section, on account of the change in the assumption of the action of the masonry invert. The stresses in the masonry section, where the sewer is constructed as a monolith

from invert to crown and the invert rests on compressible soil, are much higher and more severe in both the crown and invert, specially the latter, and even in the side walls than in the case where the side walls or invert rest on ledge foundation.

Experience with Large Sewer Section.—A few years ago the attention of the authors was called to the action of a large horseshoe sewer section which had cracked in the arch, Fig. 189. This section, although slightly smaller than the section analyzed in the foregoing discussion, was of practically the same type and was constructed as a reinforced concrete monolithic structure, on compressible soil. It will be noted that the arch cracked, as might be expected from a study of the line of resistance for Case I, where the stresses in the steel were excessive and the stresses in the concrete exceeded the ultimate strength. The locations of the cracks shown were obtained by measurement. It is probable that cracks occurred in the invert, although no definite information was obtained on account of the flow of sewage. While this structure did not fail nor was it distorted to any noticeable degree, yet the small cracks shown in the section could be easily detected and showed clearly that the steel had stretched sufficiently to allow the concrete to crack.

ANALYSIS OF 15-1/2 FT. SEMI-ELLIPTICAL SEWER SECTION BY METHOD FOR INDETERMINATE STRUCTURES

As an example of the analysis of a different type of structure from that previously shown, the following analysis of a 15 ft. 6 in. semi-elliptical type of sewer section will be of interest. The computations are given in Tables 157, 158, and 159, and the arch section, the force diagram and the equilibrium polygon are shown in Fig. 190. As the method of analysis

TABLE 157.—COMPUTATIONS OF EXTERNAL FORCES AND MOMENTS

Analysis of 15 ft. 6 in. Semi-elliptical Sewers by Method for Indeterminate Structures

1	2	3	4	5	6	7	8	9	10	11
Section number	Thickness of ring at center of section, t, ft.	t^3, ft.	ds, ft.	$\frac{ds}{t^3}$ (d_{3a}) ft.	Co-ordinates of center of section x, ft.	y, ft.	$m_x = y \times 1$ lb.	m_x^2	$m_x^2 \frac{ds}{t^3}$ (d_{2x})	$m_x \frac{ds}{t^3}$ $(d_{2a} = d_{3x})$
1	1.30	2.197	2.41	1.097	1.21	0.13	0.13	0.017	0.019	0.143
2	1.30	2.197	2.41	1.097	3.40	1.10	1.10	1.210	1.327	1.208
3	1.30	2.197	2.40	1.002	5.06	2.85	2.85	8.123	8.869	3.112
4	1.39	2.686	2.39	0.890	6.37	4.84	4.84	23.426	20.850	4.308
5	1.51	3.443	2.39	0.694	7.40	6.98	6.98	48.720	33.811	4.844
6	1.68	4.742	2.39	0.504	8.17	9.25	9.25	85.563	43.126	4.662
7	1.86	6.435	2.39	0.371	8.63	11.60	11.60	134.560	49.925	4.304
8	2.00	8.000	2.39	0.299	8.77	13.96	13.96	194.882	58.270	4.175
9	1.95	7.415	1.81	0.244	7.89	15.52	15.52	240.870	58.850	3.790
10	1.95	7.415	1.80	0.243	6.19	16.15	16.15	260.823	63.380	3.924
11	1.95	7.415	1.80	0.243	4.44	16.61	16.61	275.892	67.040	4.036
12	1.94	7.301	1.80	0.247	2.69	16.92	16.92	286.286	70.710	4.179
13	1.94	7.301	1.80	0.247	0.90	17.07	17.07	291.385	71.972	4.216
				7.268					548.149	46.901

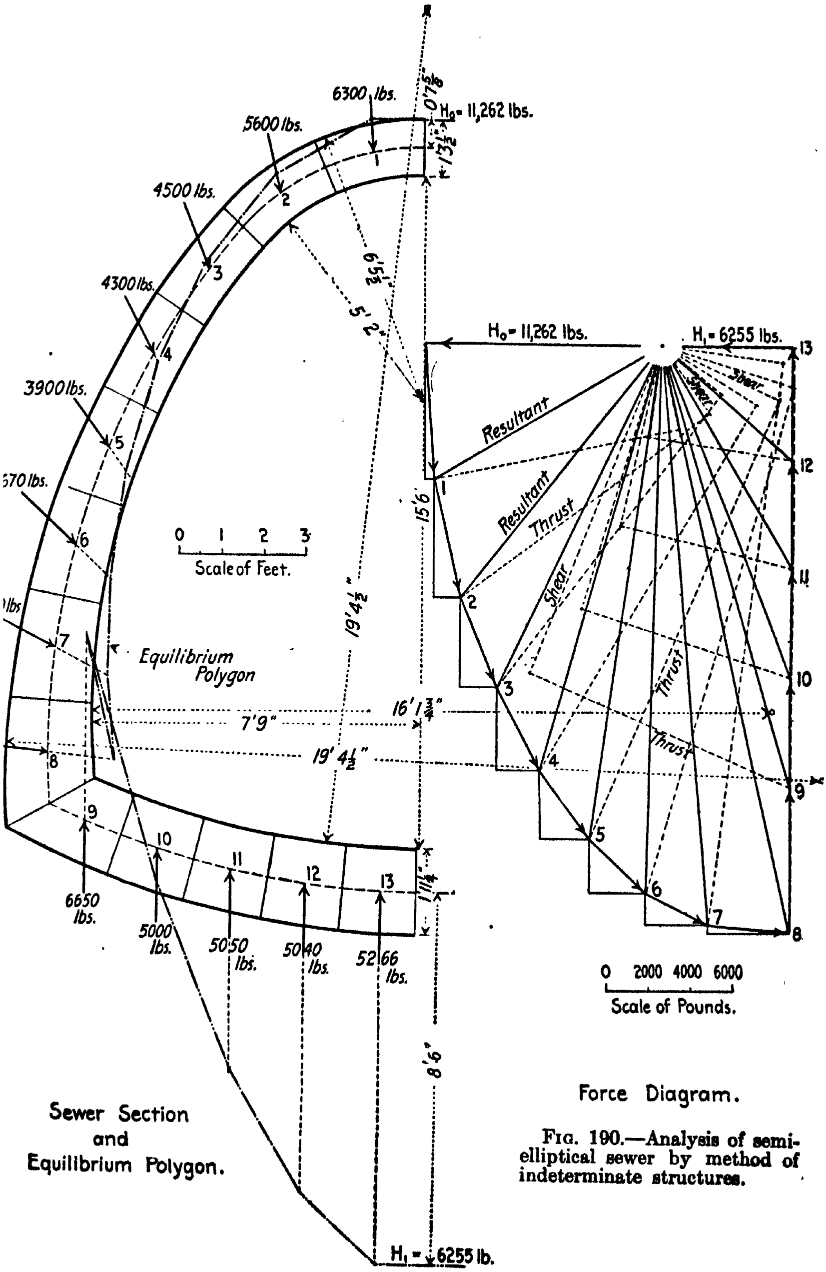


FIG. 190.—Analysis of semi-elliptical sewer by method of indeterminate structures.

TABLE 158.—COMPUTATIONS OF EXTERNAL FORCES AND MOMENTS

Analysis of 15 ft. 6 in. Semi-elliptical Sewer by Method for Indeterminate Structures

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Section number	Depth to center of extrados, ft.	Vertical intensity of earth pressure, lb per sq. ft	Horizontal projection of extrados section, ft.	Total vertical load w, lb.	Horizontal intensity of earth pressure, lb. per sq. ft.	Vertical projection of extrados section, ft.	Total horizontal load h, lb.	Sum of vertical loads Σw, lb.	Sum of horizontal loads, Σh lb.	Difference between successive co-ordinates $x_2 - x_1$, etc., ft.	Difference between successive co-ordinates $y_2 - y_1$, etc., ft.	Bending moments m, ft. lb.	$m\frac{ds}{I_s}$ Δa	$mm_x\frac{ds}{I_s}$ Δ_x
1	24.15	2,415	2.60	6,279	805	0.55	443	6,279	443	2.19	0.97	....	....	....
2	25.25	2,525	2.15	5,429	842	1.55	1,305	11,708	1,748	1.66	1.75	14,182	15,558	17,132
3	27.10	2,710	1.52	4,119	903	1.95	1,761	15,827	3,509	1.31	1.99	36,676	40,051	114,140
4	29.12	2,912	1.30	3,786	971	2.12	2,058	19,613	5,567	1.03	2.14	64,392	57,310	277,400
5	31.32	3,132	1.00	3,132	1,044	2.28	2,380	22,745	7,947	0.77	2.27	96,506	66,970	467,475
6	33.68	3,368	0.72	2,425	1,123	2.40	2,695	25,170	10,642	0.46	2.35	132,060	66,560	615,660
7	36.20	3,620	0.40	1,448	1,207	2.48	2,993	26,618	13,635	0.14	2.36	168,647	62,565	725,870
8	38.83	3,883	0.10	388	1,294	3.00	3,882	27,006	17,517	− 0.88	1.56	204,553	61,165	853,950
9	......	− 2,762	2.40	− 6,629	......	......	......	20,377	17,517	− 1.70	0.63	208,080	50,772	788,550
10	......	− 2,762	1.80	− 4,972	......	......	......	15,407	17,517	− 1.75	0.46	184,516	44,840	724,050
11	......	− 2,762	1.83	− 5,054	......	......	......	10,354	17,517	− 1.75	0.31	165,574	40,234	668,200
12	......	− 2,762	1.85	− 5,110	......	......	......	5,246	17,517	− 1.79	0.15	152,884	37,762	638,440
13	......	− 2,762	1.90	− 5,246	......	......	......	0	17,517	......	......	146,132	36,092	616,000
													579,879 $= \Delta_a$	6,506,867 $= \Delta_x$

is exactly the same as that described for the horseshoe section, no detailed explanation is necessary.

TABLE 159.—BENDING MOMENTS, THRUSTS AND SHEARS

Analysis of 15 ft. 6 in. Semi-elliptical Sewer by Method for Indeterminate Structures

1 Section	2 H_oy, ft. lb.	3 Total bending moments, M, ft. lb.	4 Thrusts, N, lb.	5 Eccentric distances, e, ft.	6 Shears, V, lb.
Crown	....	7,103	11,262	0.632	0
1	1,464	8,567	11,870	0.723	3,950
2	12,388	5,309	14,600	0.364	3,900
3	32,097	2,524	17,260	0.147	3,630
4	54,608	− 2,681	19,800	−0.136	5,250
5	78,609	−10,794	22,150	−0.486	6,140
6	104,174	−20,783	24,320	−0.855	6,580
7	130,639	−30,905	26,100	−1.190	5,800
8	157,217	−40,233	26,900	−1.500	6,350
9	174,786	−26,191	13,420	−1.950	16,350
10	181,881	4,465	10,450	0.430	12,750
11	187,062	28,591	8,350	3.430	8,550
12	190,553	44,772	6,900	6.490	4,300
13	192,242	53,213	6,255	8.500	....
C.l. inv.	192,693	52,968*	6,255	8.47	0

* Obtained in same manner as other Total Bending Moments $M = m + M_0 + H_oy_{inv.}$ or by scaling e for thrust at center of invert then $m_{inv.} = 6255 \times 8.47 = 52,980$. Use of slide rule causes slight errors.

Conditions.—The sewer section shown in Fig. 190 is of the general type shown in Fig. 151. It is assumed that the depth of earth fill over the crown of the sewer is 24 ft., that the weight of the earth filling is 100 lb. per cubic foot, and the angle of repose of the earth filling 30 deg. It is further assumed that the sewer is to be built in compressible soil without the use of piles or a timber platform.

BENDING MOMENTS (All Negative)

$m_1 = 0$
$m_2 = (6,279 \times 2.19) + (443 \times 0.97) = 14,182$
$m_3 = 14,182 + (11,708 \times 1.66) + (1,748 \times 1.75) = 36,676$
$m_4 = 36,676 + (15,827 \times 1.31) + (3,509 \times 1.99) = 64,392$
$m_5 = 64,392 + (19,613 \times 1.03) + (5,567 \times 2.14) = 96,506$
$m_6 = 96,506 + (22,745 \times 0.77) + (7,947 \times 2.27) = 132,060$
$m_7 = 132,060 + (25,170 \times 0.46) + (10,642 \times 2.35) = 168,647$
$m_8 = 168,647 + (26,618 \times 0.14) + (13,635 \times 2.36) = 204,553$
$m_9 = 204,553 + (27,006 \times -0.88) + (17,517 \times 1.56) = 208,080$
$m_{10} = 208,080 + (20,377 \times -1.70) + (17,517 \times 0.63) = 184,516$
$m_{11} = 184,516 + (15,407 \times -1.75) + (17,517 \times 0.46) = 165,574$
$m_{12} = 165,574 + (10,354 \times -1.75) + (17,517 \times 0.31) = 152,884$
$m_{13} = 152,884 + (5,246 \times -1.79) + (17,517 \times 0.15) = 146,132$
$m_{inv.} = 146,132 + (0 \times -0.90) + (17,517 \times 0.04) = 146,833$

$$H_o = \frac{579{,}879 \times 46.901 - 6{,}506{,}867 \times 7.268}{46.901 \times 46.901 - 548.149 \times 7.268}$$

$$H_0 = +\ 11{,}262.$$

$$M_0 = \frac{579{,}879 \times 548.149 - 6{,}506{,}867 \times 46.901}{7.268 \times 548.149 - 46.901 \times 46.901}$$

$$M_0 = +\ 7{,}103$$

$$e = \frac{M_o}{H_o} = \frac{7{,}103}{11{,}262} = +\ 0.632$$

COMPUTATION OF STRESSES IN ARCH SECTION

In the previous discussions the thrust, shear and bending moment have been computed for the various sections of the arch ring. The next step in the design of the sewer arch is to determine the maximum stresses in the masonry or steel in order to make sure that the actual stresses do not exceed the safe working stresses and, further, to determine that the section has been designed as economically as possible.

As already stated, the shear can usually be neglected in concrete and reinforced concrete arches, for concrete is relatively strong in resisting shear and the arches usually employed in sewerage practice do not develop high stresses in shear.

In order to simplify the discussion, plain concrete or masonry sections will be considered separately from concrete sections reinforced with steel. The following discussion and formulas have been taken by permission from Taylor and Thompson, "Concrete, Plain and Reinforced," 2nd Edition.

Let R = resultant of all forces acting on any section,
f_c = maximum unit compression in concrete,
f_c' = minimum compression in concrete,
N = thrust, the component of the force normal to the section,
V = shear, the component of the force, R, parallel to the section,
b = breadth of rectangular cross-section, taken as 12 in.,
t = thickness or height of rectangular cross-section,
e = eccentricity, that is, the distance from the axis to the point of application of the thrust, which is the intersection of the line of pressure with the plane of the section,
M = bending moment on the section,
f_s' = maximum unit compression in the steel,
f_s = maximum unit tension or minimum unit compression in the steel,
p = ratio of steel area at both faces to total area of section,
= for rectangular sections, ratio of steel area to bt,
n = E_s/E_c = ratio of moduli of elasticity of steel and concrete,
k = ratio of depth of neutral axis to depth of section t,
kt = distance from outside compressive surface to neutral axis,
d' = depth of steel in compression,
d = depth of steel in tension,
a = distance from center of gravity of symmetrical section to steel,
e_o = value of eccentricity which produces 0 stress in concrete at outer edge of rectangular section opposite to that on which thrust acts.
C_a, C_s = constants.

Stresses in Plain Concrete or Masonry Arch Section.—Sewer arches constructed of plain concrete or masonry should be so designed that the line of resistance will not lie outside of the middle third of the section at any point. It is assumed in the design that plain concrete, brick or stone masonry cannot resist tensile stresses, and on that account the line of resistance should lie within the middle third, so that there will be nothing but compressive stresses developed.

The general formulas for the compressive stresses, both maximum and minimum, in any section of the arch ring, are as follows (see Fig. 191*a*):

$$\text{Maximum} = f_c = \frac{N}{bt}\left(1 + \frac{6e}{t}\right)$$

$$\text{Minimum} = f_c' = \frac{N}{bt}\left(1 - \frac{6e}{t}\right)$$

These general formulas apply to rectangular sections and will hold as long as the safe tensile strength of the concrete or masonry is not exceeded. As previously stated, however, no tension should be allowed to exist in the

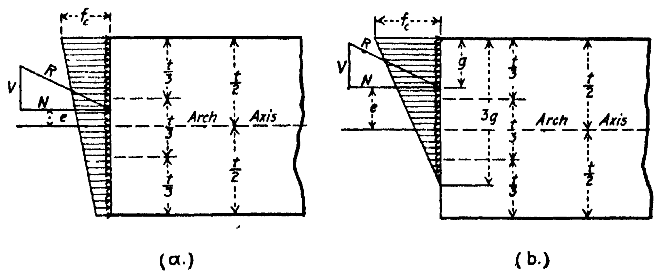


FIG. 191.—Stresses caused by forces acting on plain concrete section.

masonry. In the examination of arches already constructed, it sometimes happens that the line of resistance is found to be outside of the middle third and since it is assumed that the material is unable to carry tension, the preceding formula is not applicable for computing the stresses on the section. In this case the stress is distributed as compression over a depth less than the entire depth of the section and cracks may be expected on the tension side (see Fig. 191*b*). The maximum compression in this case equals

$f_c = \frac{2N}{3bg}$ where g = distance from point of application of thrust to most compressed surface.

Stresses in Reinforced Concrete Section.—In reinforced concrete sections the area of steel in compression can be replaced in the design by an equal area of concrete by multiplying the steel area by n, the ratio of modulus of elasticity of steel to the modulus of concrete. The moments of inertia may also be compared in a similar manner and the section treated in the design as if it were entirely composed of concrete. In the design of a reinforced concrete section it is assumed that the concrete is not allowed to carry tension, but that all of the tensile stresses must be carried by the steel reinforcement.

No Tension in Section.—The following equation expresses the value of the maximum unit compression in the concrete under conditions where no tensile stresses exist in the section (see Fig. 192).

$$f_c = \frac{N}{bt}\left[\frac{1}{1 + np} + \frac{6te}{t^2 + 12npa^2}\right]$$

This condition does not necessarily mean that the line of pressure lies at or within the limits of the middle third of the section, for in a reinforced concrete section the value of the eccentricity, e_o, at which there is neither compression nor tension at the surface opposite to that on which the thrust acts is usually somewhat greater than $t/6$. For greater values of the eccentricity than e_o, and assuming that the concrete is unable to carry any tension, the above formula is not applicable.

For convenience, the above formula may be expressed as follows:

$$f_c = \frac{NC_e}{bt} \text{ where } C_e = \left[\frac{1}{1 + 15p} + \frac{e}{t}\,\frac{6}{1 + 28.8p}\right]$$

based on the assumptions that $n = 15$ and $2a = \frac{4}{5}t$, which are reasonable and can be used for most all cases without great error.

In the diagram, Fig. 193, are given values of C_e for various values of e/t and different percentages of steel. The curve in the lower right corner is plotted to give values of e_o/t or different percentages of steel, and is useful for determining whether or not a given eccentricity will produce tension in the section. For example, if the thickness of the arch section is 18 in. and the percentage of steel reinforcement is 0.8 ($p = 0.008$) from the curve $e_o/t = 0.183$, and therefore $e_o = 3.29$ in. This means that the line of pressure or point of application of the thrust cannot be more than 3.29 in. from the arch axis without producing tension on one side.

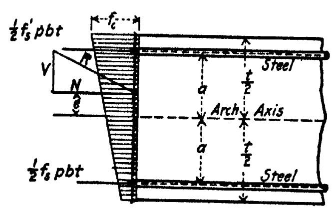


FIG. 192.—Stresses caused by a force producing compression upon the whole reinforced section.

To illustrate the use of the curves for C_e if in the above example the eccentricity is 2 in. $e/t = 2/18 = 0.111$, and $C_e = 1.44$. Therefore $f_c = 1.44N \div (12 \times 18)$, from which the value of f_c can be found if the thrust N is known.

If tension does not exist in the section, the principal stress to be determined is the maximum compression in the concrete which must not exceed a safe working stress.

Tension in Section.—When the eccentricity is greater than e_o and the concrete is considered as unable to carry tension, the following formula should be used (see Fig. 194):

$$f_c = \frac{M}{C_a bt^2}$$

where

$$C_a = \left[\frac{npa^2}{t^2k} + \frac{k}{4} - \frac{k^2}{6}\right]$$

To facilitate the computations, Fig. 195 is given. Determine e/t and from the left-hand diagram find the corresponding value of k for the given percentage of steel. Then with this value of k use the right-hand curves to find the corresponding value of C_a for the given percentage of steel.

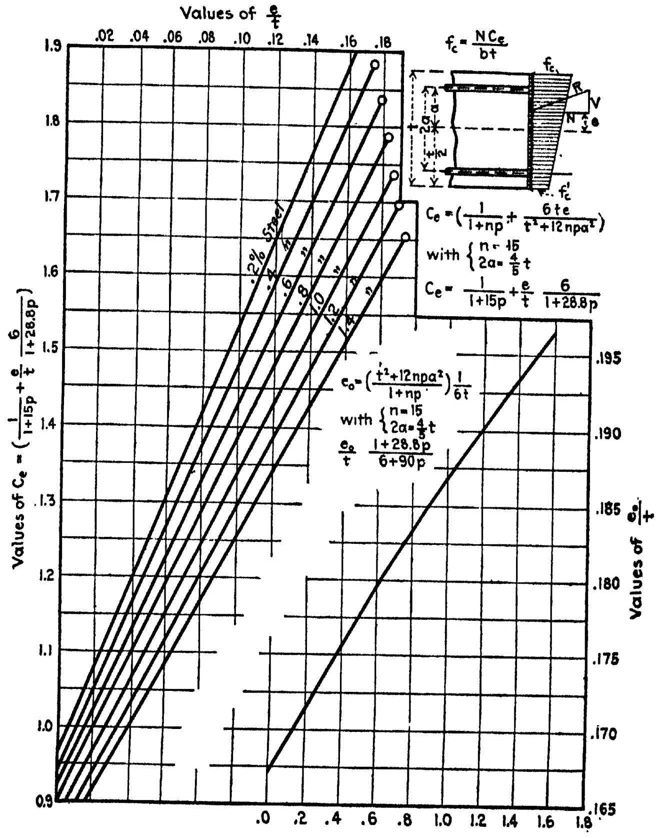


FIG. 193.—Diagram for determining compression and eccentricity.
$n = 15$ and $2a = \frac{4}{5}t$.
(Reproduced by permission of the authors from "Concrete, Plain and Reinforced," second edition, by Taylor and Thompson).

For illustration, if in the example previously given the value of e is 10 in., then $e/t = 0.56$ and from Fig. 195, $k = 0.46$, and the steel percentage 0.8 per cent. as before, $C_a = 0.1215$. Then $f_c = M \div (0.1215 \times 12 \times 18 \times 18)$ from which the value of f_c can be found if the bending moment M is

known. It should be borne in mind that if e and t are expressed in inches, the values of M should be in inch-pounds.

Having thus found the unit stress in the concrete, the unit stresses in the steel may be found by the following formulas (see Fig. 194);

$f_s' = nf_c\left(1 - \frac{d'}{kt}\right)$ = maximum unit compressive stress in steel.

$f_s = nf_c\left(\frac{d - kt}{kt}\right)$ = maximum unit tensile stress in steel.

Shearing Stress.—This is found as follows:

Let V = total shear at any section,
v = maximum unit shearing stress,
b = thickness of section assumed = 12 in.,
jd = arm of resisting couple = approx. $\frac{7}{8}d$,
then $v = \frac{V}{bjd}$ = (approx.) $= \frac{8V}{7bd}$.

As a rule the shearing stress may be neglected, but in the case of one or two critical joints subjected to a large shear it should be computed. The above formula may be used.

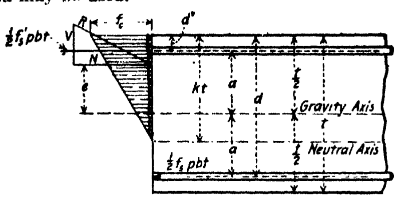


FIG. 194.—Stresses caused by a force producing compression and tension upon a reinforced section, tensile strength of concrete neglected.

Bond Stress.—This is computed as follows:

Let u = unit bond stress between concrete and steel bars,
o = perimeter of one bar,
Σo = sum of perimeters of bars in unit length,
then $u = \frac{V}{\Sigma ojd}$ = (approx.) $= \frac{8V}{7 \times \Sigma od}$

The above formula can be used when necessary to compute the bond stress, but as a rule this computation can be omitted.

TRANSVERSE STEEL REINFORCEMENT

The fact has already been pointed out that the introduction of steel reinforcing bars to strengthen the arch where only compressive stresses exist does not permit of any great diminution of the concrete section or any marked economy, but it does have the

great advantage of making the structure more reliable and acts as a sort of insurance against unforeseen stresses which are liable to occur, such as stresses due to temperature changes or shrinkage of the concrete, settlement of foundations, and the like. It also provides an additional factor of safety against poor workmanship in the construction of the sewer section. While the designer may make an effort to foresee the conditions and to provide sufficient reinforcement or thickness of masonry to withstand the stresses as computed, there is an uncertainty concerning the action of arches for which it is impossible wholly to provide.

In view of these considerations, it is well to use transverse steel reinforcement for large concrete sewers, even though the computations may show that the line of resistance lies everywhere within the middle third of the masonry section. It is impossible in arch reinforcement to make use of the steel to the full allowable compressive working stress used in steel design. The maximum compressive stress which can be reached in a reinforced concrete arch designed in accordance with the foregoing method of computation, will never be greater than the allowable working stress in the concrete multiplied by the ratio of the moduli of elasticity n. This, under ordinary conditions, places a limit in compression on the steel reinforcement of approximately 7500 lb. (500 × 15) per square inch. If a greater compressive stress should be developed in the steel the deformation would be sufficiently great to overstress and crush the concrete.

In good practice the amount of

TABLE 160—COMPUTATION OF STRESSES

Analysis of 15 ft. 6 in. Semi-elliptical Sewer by Method for Indeterminate Structures

1	2	3	4	5	6	7	8	9	10	11	12	13	14
			Assumed steel reinforcing							Maximum unit stresses in lb. per sq. in.			
Section number	$\frac{e}{t}$	Area of concrete, sq. in.	Size of bars, in.	Spacing, in.	Area of steel at tension face, sq. in.	Ratio of area of steel to area of concrete	k	C_a	$\frac{M}{bt^2}$	Compression in concrete, f_c	Tension in steel, f_s	Shearing stress, v	Bond stress, u
Crown	0.490	187.2	$\frac{7}{8}$	24	0.77	0.0041	0.42	0.099	29.2	295	5,100	....	.....
6	0.508	242.0	$\frac{7}{8}$	24	0.77	0.0032	0.39	0.096	50.8	530	10,790	....	.....
8	0.750	288.0	$\frac{7}{8}$	6	3.06	0.0106	0.42	0.136	70.0	515	9,500	....	.....
9	1.0	280.0	$1\frac{1}{8}$	6	5.06	0.0181	0.45	0.175	48.2	275	4,400	73	73
c invert	4.36	279.0	$1\frac{1}{8}$	6	5.06	0.0181	0.34	0.195	97.9	503	13,200	....	.....

transverse reinforcement in arches usually varies from about 0.2 to 1.5 per cent. of the area of the concrete masonry at the crown.

In designing the reinforcement for a sewer arch, it is necessary to assume a certain percentage of steel at the start, as will be noticed from the method of computing fiber stresses, already given. After the computations have been made the actual percentage to be used can be adjusted in accordance with the results of the computation, in order to obtain the most economical arrangement possible.

Computation of Transverse Reinforcement for 15 ft. 6 in. Semi-elliptical Section.—As an example of the method of computing the re-

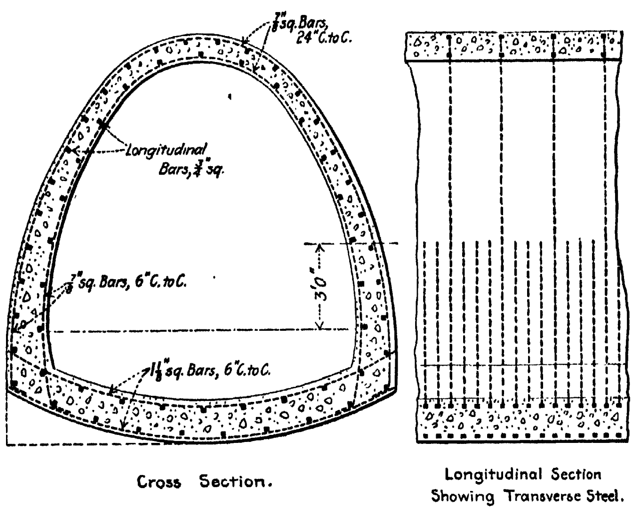


Fig. 196.—Steel reinforcement of 15 1/2-ft. semi-elliptical sewer.

inforcement the following computations, Table 160, made for the 15 1/2 ft. semi-elliptical section previously analyzed, are given. As a rule it is not necessary to compute the stresses for each division, but merely for a few critical points.

It is customary to keep the same size of bars and the same spacing in the upper part of the arch, changing either or both if necessary in the side-walls or in the invert. It is desirable to have as few different sizes of bars as practicable. In general, smaller bars closely spaced are preferable to larger bars with wide spacing. A typical arrangement of the transverse steel reinforcing bars is shown for the 15 ft. 6 in. semi-elliptical sewer in Fig. 196.

It will be noted that the shearing stress on division 9 is higher than commonly allowed. While there is some question concerning the necessity of keeping the shearing stress at this particular location within the usual allowable limits, this can be done by increasing the concrete in the invert as shown in Fig. 196 by the dotted line on the left side.

LONGITUDINAL STEEL REINFORCEMENT

Masonry structures of all kinds expand and contract with temperature changes. This is especially noticeable in concrete structures, for the cracks are more readily seen than in stone or brick masonry structures where the cracks are distributed among so many joints as to be practically invisible. Concrete conduits or sewers are subject to temperature changes, particularly during the period of construction. The expansion of the masonry rarely causes trouble except at sharp angles, but contraction is more likely to cause difficulties.

Two methods are in use for preventing objectionable cracks caused by the shrinkage of concrete in hardening and the contraction due to a lowering of the temperature. One method is to locate expansion joints at frequent intervals, approximately 30 ft., so that all of the changes will be concentrated in one crack at each expansion joint. The second method is to insert enough reinforcement composed of small bars placed near the surface of the concrete to distribute the cracks at short intervals and make them so small as to be practically invisible or unobjectionable. In actual practice it has been customary to insert from 0.2 to 0.4 per cent. of the area of the concrete as longitudinal steel to resist shrinkage and temperature stresses. For this purpose deformed bars furnishing a high mechanical bond with a high elastic limit are advantageous.

It is interesting to note that concrete laid during warm weather is much more likely to crack on account of temperature changes than concrete laid during cold weather, and in addition, shrinkage cracks are more apt to occur with concrete laid during hot, dry weather unless care is taken to keep the concrete wet.

The actual amount of steel reinforcement to be provided to resist temperature stresses is, to a certain extent, a matter of judgment. For a sewer constructed in comparatively dry soil and designed to carry both surface water and sewage, the presence of small cracks might be considered unobjectionable. Large cracks would doubtless be objectionable on account of the possible rusting of the steel reinforcement and consequent weakening of the structure. For a sewer constructed in very wet soil adjacent to a river or a creek, where it is essential to keep out as much ground water as possible, the presence of even small cracks might be objectionable.

Taylor and Thompson, in "Concrete, Plain and Reinforced," second

edition, page 501, give the following formula, suggested by Charles M. Mills, for estimating the size and distance apart of cracks, so as to form a basis for judgment as to the sizes and percentages of steel to use. Let x = distance apart of cracks, D = diameter of round bar or side of square bar, p = ratio of cross-section of steel to cross-section of concrete. Assuming that the strength of concrete in tension is equal to the bond between plain steel bars and concrete the distance apart of cracks is $x = D/2p$ for square or round bars. If instead of plain bars deformed bars are used, having twice the bond strength of plain bars, the cracks would be one-half as far apart and only one-half as wide.

Taylor and Thompson also suggest that the size of the crack is governed by the amount of shrinkage and on that account the size may be estimated as the product of the coefficient of contraction (0.0000055) by the number of degrees fall in temperature, by the distance between cracks.

If it is desired to prevent the appearance of cracks so far as possible, and to make the sewer practically watertight 0.4 per cent. of steel should be used, that is, the ratio of the area of the steel to the area of concrete should be 0.004.

The presence of longitudinal reinforcement also has the advantage of making it possible to tie both the transverse and longitudinal bars together and thereby aid in the erection of the steel. Where the two sets of bars are wired together at frequent intervals there is also less likelihood of their becoming displaced during the placing of the concrete. In fact, if no longitudinal reinforcement is used on account of temperature and shrinkage stresses, it will be advisable to use a certain number of longitudinal bars to support and space the transverse bars. While this is not absolutely necessary, it can be done at a slight expense and is justified by the greater certainty of having the bars located in their proper places.

The following computations will serve to illustrate the application of the foregoing discussion:

Assuming that the amount of longitudinal steel reinforcement to be provided for the 15 1/2 ft. semi-elliptical section previously analyzed, is 0.25 per cent. or $p = 0.0025$, and that ¾-in. plain square bars are to be used, the distance apart of the cracks would be $x = D/2p = 0.75/2 \times 0.0025 = 150$ in. Further assuming that the maximum change in temperature of the concrete masonry may be 50°, the width of the crack will be $0.0000055 \times 50 \times 150 = 0.0412$ in.

If, on the other hand, deformed steel bars were to be used with a bond strength 50 per cent. greater than that of plain bars, which is a reasonable assumption, the spacing of the cracks will be inversely proportional to the unit bond of the steel bars. In this case the spacing

of the cracks would be 100 in. and the width of a crack would be 0.028 in.

The area of the concrete section for the 15 1/2 ft. semi-elliptical sewer is 13,576.9 sq. in.; 13,576.9 × 0.0025 = 33.94 sq. in. of steel bars for longitudinal reinforcement. Area of 3/4 in. square bar = 0.5625 sq. in.; 33.94/0.5625 = 60 bars. These bars are distributed as shown in Fig. 196 so as to reinforce the interior and exterior surfaces approximately uniformly.

SAFE WORKING STRESSES

The working stresses recommended by the Joint Committee on Concrete and Reinforced Concrete (*Proc. Am. Soc. Test. M.*, vol. xiii, p. 270), furnish the best guide for determining safe values to use in design. For a complete understanding of the following figures, reference should be made to that report. The following working stresses for concrete are based on the assumption that concrete composed of 1 part of Portland cement and 6 parts of aggregate is capable of developing an average compressive strength of 2000 lb. per square inch at 28 days when tested in cylinders 8 in. in diameter and 16 in. long under laboratory conditions of manufacture and storage, using the same consistency as is employed in the field.

	Lb. per sq. in.
Compression on extreme fiber not over..................	650*
Shear and diagonal tension not over......................	40
Bond...	80
Tensile stress in steel not over..........................	16,000

The above figures for concrete should be reduced if the concrete has an average strength less than that specified.

UNSYMMETRICAL LOADING

A direct determination of the stresses in a masonry arch, loaded unsymmetrically by the voussoir method described in Baker's "Masonry" is impossible, but a solution can be arrived at by approximate methods.

The elastic theory of the arch permits a direct determination of the stresses for unsymmetrical loads, but the labor is greatly increased over that indicated in the preceding analyses.

* It is important to notice that these figures are for a 1:6 mixture and must be modified for other mixtures as explained in the Joint Committee's report. The authors' practice is to use 500 lb. per square inch maximum compression in the extreme fiber, 40 lb. maximum shear where only horizontal reinforcement is used, 50 lb. maximum shear with horizontal bent-up bars, 80 lb. maximum shear with horizontal bent-up bars fully supplemented with stirrups, 500 lb. maximum bearing strength, 64 lb. bond stress for plain bars, not including drawn wire, and 130 lb. bond stress for deformed bars.

Except in unusual cases and for very wide span sewer arches it is seldom necessary to compute the stresses due to an unsymmetrical load. If the conditions of unsymmetrical loading are sufficiently severe to warrant a special analysis, the elastic theory may be used.

DETAILS

Curves.—Changes in direction of large sewers should always be made by curves. It is impossible to give an exact statement for the proper radius of a curve for any particular size of sewer, but various approximate methods have been used and found to produce fairly good results. The best discussion is by W. E. Fuller in *Jour. N. E. Water-Works Association*, December, 1913.

W. W. Horner, in *Engineering and Contracting*, Sept. 13, 1911, states that in the St. Louis Sewer Department the practice has been to make the radius of the curve as large as possible, varying from 30 to 80 ft. when in street intersections and from this up to a 2 deg. curve where the angle is small.

On the Louisville, Ky., sewers constructed about 1908 to 1912, the radii on curves have varied from 16 to over 400 ft. for sewers from 5 to 14 ft. in diameter. The major part of the curves, however, were constructed with radii from 30 to 50 ft. in length.

Some compensation should be made for the loss in head due to increased friction on curves. A method of making such compensation has already been outlined in Chapter III. The formula given is that offered by P. J. Markmann of the St. Louis Sewer Department.

In *Trans. Am. Soc. C. E.*, December, 1905, Walter C. Parmley states that in the design of the Walworth sewer in Cleveland, Ohio, all changes in direction of the main sewer were made with as easy curves as possible. At one intersection where the deflection was about 90 deg. two lots were purchased and the sewer was built on a curve of 164 ft. radius.

Several authors suggest that the additional loss due to sharp curves be assumed as $0.5v^2/2g$ where v is the mean velocity and g is the acceleration of gravity. Other designers arbitrarily allow a certain amount of fall between the beginning and end of the curve, the amount of such increase being selected by judgment for the particular case.

Changes in Size.—It is a well-established fact that abrupt changes in the size of a waterway, such as sudden enlargement or sudden contraction cause increased friction and consequent loss in head. If the section is enlarged gradually, this loss can be practically eliminated. The proper length in which changes in size should be made has usually been selected by judgment.

Hughes and Safford, "Hydraulics," suggest that a batter of 1:10 on the sides of the sewer will be found favorable. This means that the diameter of the sewer is increased 2 ft. in 10 lin. ft.

CHAPTER XIV

STREET INLETS, CATCH-BASINS AND MANHOLES

The special structures which are built on sewerage systems have an important part to play in the operation of such works, as a rule. In order to clean sewers, manholes giving access to them are provided, and drop-manholes and wellholes have been developed from ordinary manholes, in order that sewage may be delivered vertically from one elevation down to another with a minimum amount of disturbance. For this latter purpose flight sewers, with their inverts like a straight stairway, have also been constructed. Where storm-water is removed underground, street inlets are provided to discharge it directly into the sewers and drains, and catch-basins are employed where this surface run-off contains so much refuse of different kinds that the engineer prefers to give it a chance to settle in a readily-cleaned sump rather than to allow everything to flow without check into the sewers. In order that long lines of small sewers may be kept under observation with the greatest facility, some engineers provide them with lampholes, down which a lamp can be lowered to illuminate the interior of the sewer enough to enable an observer at the manhole on either side of the lamphole to see with more or less distinctness the condition of the pipe.

There are many small sewers with grades so flat that the only way to keep them clean is to flush them with water, accompanied if necessary by scrubbing with a brush on the end of a long rod or wire. For this purpose a flushing manhole operated manually or an automatic flush-tank is employed, and there is a great difference of opinion among engineers regarding the respective merits of the two types. Occasionally a flushing inlet is provided on the bank of some river or pond, through which water can be admitted to large sewers which need cleaning.

Where large sewers join together there are bellmouths and other forms of junctions to be built, which sometimes assume forms of considerable complexity. Inverted siphons are used in crossing valleys or dropping below subways and other obstructions. On rare occasions a true siphon may be used to overcome a small ridge, although it is usually considered preferable to go to considerable expense to avoid such a detail. Since reinforced concrete came into use, specially designed hollow girders or beams have been employed in some places to cross rivers or deep gulches, where inverted siphons or steel bridges would have been used before. If the combined sewerage system in-

cludes intercepting and relief sewers, some form of regulating device must be used at each place where the sewage is discharged from a collecting sewer into an intercepting or relief sewer; there are numerous forms of automatic regulators, storm overflow chambers and leaping weirs used for such situations.

Where the sewage is discharged into a river, lake or tide water, an outlet of some kind is needed; it has already been pointed out in the Introduction, that the failure of the designers of early sewerage systems to allow for the effect of tide-locking of such outfalls caused a large part of the really serious troubles with some of the sewerage systems built prior to about 1875. Even today the effect of submergence on the flow in an outfall sewer and on the discharge from its outlet is not always given the attention it requires. Another allied type of special structure is the tide gate, which is a large check-valve to prevent the entrance of water into a sewer when its surface elevation reaches such a height that the water tends to pass in through the valve rather than the sewage pass out.

In the early days of sewerage works, their ventilation received a large amount of attention and a great variety of theories existed concerning the best way to carry this out. The omission of the main house trap was advocated by some engineers as a material aid in sewer ventilation, because of the upward draft through the soil pipes of the buildings which it was claimed would come into existence in this way. Another body of engineers vigorously opposed the omission of the main trap and insisted upon a vent pipe run from the house drain, outside the trap, up the side of the building to an outlet above the highest windows. Still other engineers made use of ventilating chimneys shaped like the posts of street lamps, and sometimes used as such, and at one time perforated manhole covers were in quite general use as a means of ventilation. Taking it all in all, it is perhaps safe to say that there has been no part of sewerage engineering in which a greater variety of special designs has been prepared for the same purpose than in ventilation, while the vigor of the debates over it down to the last decade of the last century was a noteworthy feature in the engineering literature of the day.

Although some of these special structures offer no opportunity for standardization, for the local conditions of each case are different, in each class there are certain features which experience has indicated are important. In rare cases, experience has shown further that some details will not be satisfactory in service. Little has been done as yet toward a really thorough co-operative study of these special structures by engineers in different cities, but a little has been accomplished by correspondence and visits between engineers interested in some of the details. In the following notes, the authors' purpose is to describe structures which will illustrate not only types but also the preferences of

a number of engineering offices. Experience rather than theory must control the design of many such details, and if the experience of an engineering office with its standard for any detail has been satisfactory, no change should be made from that detail without careful consideration. While standardization will gradually take place, the rate of progress will inevitably be very slow, as is always the case in advances depending upon individual experience, unaided by the publicity which promoters and salesmen give to the things they are introducing.

STREET INLETS AND CATCH-BASINS

The storm water which remains on the surface of the ground, instead of percolating into it, and must be removed through the sewerage or drainage systems, is collected in the street gutters which convey it to inlets. These inlets are either the ends of direct connections to the sewers, or else discharge the storm water into catch-basins provided to intercept the refuse which the water has carried from the street surfaces in its course to the inlet. It is evident, therefore, that the location of these inlets is a matter of importance to the authorities in charge of the streets as well as those who are connected with the sewer department, for it is manifestly important to keep the streets free from water and the gutters in such condition, even during a heavy rainstorm, that it is possible for teams to drive close to the curb and for pedestrians to cross the street with the minimum inconvenience. A little consideration will show that there can be no fixed rules governing the location of the inlets, if the convenience of the public is to be served most effectively. The topography of a city often tends to concentrate the run-off of storms in certain places, and it is the duty of the sewerage engineer to prevent this concentration so far as practicable. This can only be done by intercepting the storm water as it flows through the gutters at the higher elevations, and to accomplish this in the best way the street department may be very properly requested to depart at times from some of its standard regulations regarding curbing and gutters. The street department may have good reasons for refusing to allow any sudden drop in the grade of a gutter at an inlet, for such quick depressions, even of a depth of 0.5 in., invite an early disintegration of the material of the gutter at that place. This is not true, however, if the depression is made rather gradually, and there is no valid reason for objecting to such a depression in the gutter in order to give a depth of curb on a side-hill street which will permit the construction of an inlet of sufficient capacity to care for the storm water that should be intercepted. These statements are made at the outset of the discussion of street inlets and catch-basins, because the lack of co-operation between street and sewer departments has been the cause of some unsatisfactory

design in the past. A good comment on the situation as it exists in many cities—recently received from W. W. Horner, of the St. Louis sewer department—reads as follows:

"I think the use of standard inlets at standard locations, without regard to the work required of them, is the most common fault in sewer design. The street pavement officials usually demand that there shall be no break in the curb line at an inlet and no great depression in the pavement or gutter. Under these conditions inlets on steep streets, other than those at the foot of the grade, are almost useless. Our standard opening in the curb is 4 ft. long and 8 in. high, and only a small proportion of the height, 2 to 4 in., is below the normal gutter line. The only solution seems to lie in the multiplication of these openings, two to four in a series, and a continuous basin behind the curb and under the sidewalk."

Unfortunately the sewerage engineer rarely has anything to say concerning the grades and cross-sections of the gutters in the streets. The public suffers from this, because at those places where it is most important to keep the streets free from water, that is to say, in the districts where there is heavy travel on the pavement, the proper location of inlets is most important and the street department generally solves it by allowing the use of as few inlets as possible, since they are an undoubted interference with the most satisfactory execution of curb and gutter construction. Wherever a street inlet exists in such a crowded thoroughfare, it is a more than even chance that there will be some defect in the pavement, due to the passage of wheels over the inlet castings or the stone sills which are sometimes used instead of castings. Nevertheless, it is the convenience of the public which must be considered in such cases, and that convenience demands that there shall be an ample number of these inlets located where they are most needed.

This location is very difficult to obtain if determined by anything except the exercise of good judgment. Experience shows that in a given city gutters of a given cross-section and slope will care for the run-off of districts of certain sizes, and that larger districts will cause the gutters to be over-filled. This information, which can only be obtained by observation during a number of years, is not always available. No one can furnish such information to the designing engineer, and he must proceed on the assumption that inlets should never be more than about 300 to 350 ft. apart where gutters should carry only a small amount of water, and never more than about 700 ft. apart, and that where two grades join to form a valley, an inlet must always be placed in the valley on the side of the street. The gutters should be so constructed, the cross-section of the street should be so selected, and the inlets so placed, that storm water will never flow across the pavement in order to reach an inlet. In rare cases a gutter may be connected with another on the opposite side of the street by a culvert, but such a culvert should be carefully designed

so that it can be kept clean and free from water which will afford a breeding place for mosquitoes. On straight grades the inlets are placed at the street corners. Although it is customary in many places to locate the inlets at the angle of the corner, this is a poor place for them if the travel on the street is more than moderate, for the wheels of trucks rounding the corner close to the curb are particularly hard on both pavement and inlet casting in such a position. If the grades are steep an inlet on each side of the corner, just before the cross walk is reached, offers the best solution of the problem in most cases. In case of doubt it is well to remember that the convenience of the public is better served by having too many rather than too few inlets. What has been said applies equally well to street inlets and to catch-basins, although there is considerable difference between these two classes of structures.

Street Inlets.—Since a street inlet affords a direct connection between the gutter and the sewer, it is very important that it should be so designed that as little opportunity as possible exists for its stoppage. The obstruction may arise through the clogging of the opening (mouth or gully) by which the water enters, or it may occur in the trap if it has one, like *A* in Fig. 197, or it may occur in the pipe running to the sewer. The objects which cause the most trouble at the openings of the inlet are sticks, waste paper, and leaves. If sticks become lodged against the opening the leaves and waste paper drawn to it by the next flush of storm water are likely to cause a stoppage. To avoid this some engineers have tried the use of openings presenting hardly any obstacle to the entrance of these three classes of refuse, but it seems questionable whether it is safe to allow sticks, at least, to enter the street inlet, owing to the danger of stoppage of the pipe leading from the opening to the sewer.

As a general proposition, it is probable that street inlets are better adapted for busy streets with good pavements which are kept clean, particularly where there are no steep grades nor any topographical conditions tending to concentrate the storm run-off at a few points, than they are for streets furnishing large quantities of refuse rarely removed by street cleaning, and liable to have the run-off concentrated at a number of places to which many storms are certain to take a large amount of street litter of every sort.

If the sewers in a district are on self-cleansing grades except at a few points, it may be best to construct grit-chambers in the sewers near these places in order to keep down the expense of maintenance by forcing most of the grit to gather in pits whence its removal will be less expensive than from the sewers of low grade.

In 1913 a number of standard types of street inlets were adopted by the president of the Borough of the Bronx, New York City. These are shown in section in Fig. 197. Type *A* has an opening 7 in. high and

2 ft. 8 in. long, in the curb. The box of the inlet is 3 ft. 5 in. × 3 ft. 6 in. × 5 ft. 6 in. deep, inside dimensions. The 12 in. vitrified pipe leading from it has a vitrified cover through which the pipe can be cleaned if it should become stopped. The quarter bend is so placed, it will be observed, that the inlet is actually turned into a diminutive catch-basin. In type *B* the inlet has a box 3 ft. 5 in. × 2 ft. 8 in. × 4 ft. 6 in. deep. The opening in the curb is 7 in. high and 2 ft. 8 in. long. This type has a 12-in. sump below the vitrified outlet, but lacks the water seal of type *A*. Type *C* has a box circular in plan 2 ft. 6 in. in diameter and 18 in. deep. In order to give it sufficient receiving capacity

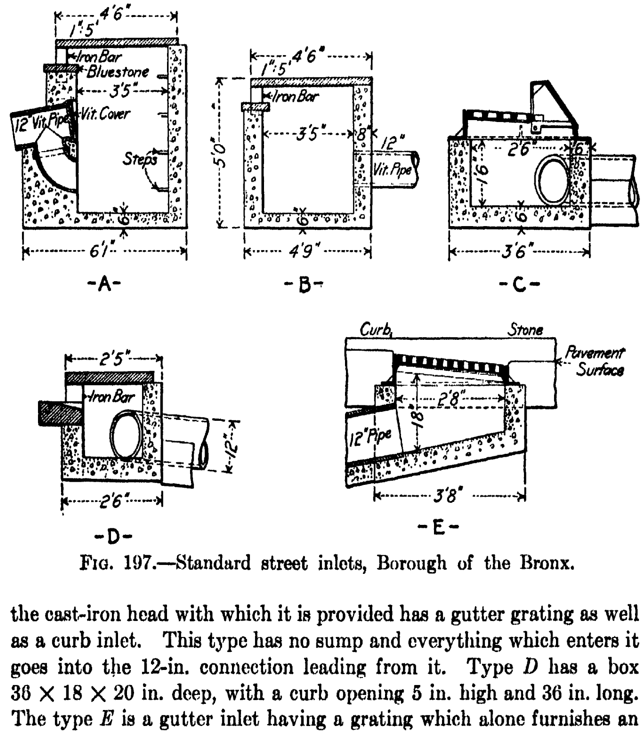


FIG. 197.—Standard street inlets, Borough of the Bronx.

the cast-iron head with which it is provided has a gutter grating as well as a curb inlet. This type has no sump and everything which enters it goes into the 12-in. connection leading from it. Type *D* has a box 36 × 18 × 20 in. deep, with a curb opening 5 in. high and 36 in. long. The type *E* is a gutter inlet having a grating which alone furnishes an inlet to the connection. The box of this inlet is 14 in. wide and its depth varies as shown in the illustration.

A type of inlet which the authors have found very satisfactory in their work is illustrated in Fig. 198. It has the advantages of durability, imparted by the substantial concrete block within which the channel is

formed, relatively low cost, a large grate opening, and easy construction. The authors' experience with inlets having risers of straight pipe is that the gratings do not have adequate openings, and the use of a reducer is very desirable in order to gain room for a larger grating.

In Philadelphia a standard inlet has been adopted which is constructed wholly of concrete, brick and flagging, as shown in Fig. 199,

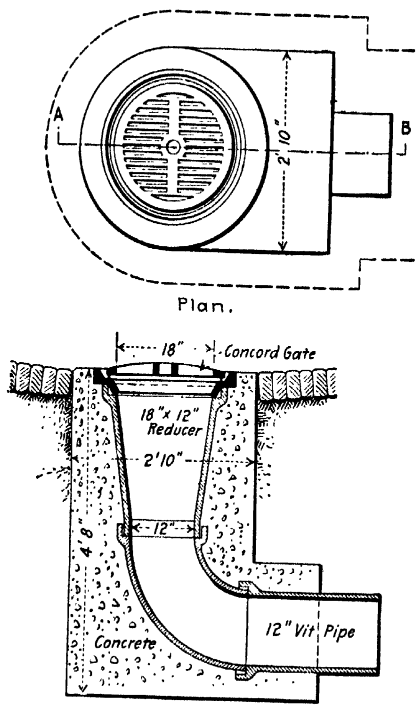


FIG. 198.—Standard inlet, Metcalf and Eddy.

which shows two variations of the type, one with a curb and the other with a gutter opening.

Catch-basins.—The catch-basin was formerly considered an absolutely essential part of any American combined sewerage or drainage system. Experience had shown that the velocity of the sewage flowing in many sewers was insufficient to prevent the formation of beds of sludge within the sewers, and it was manifestly more expensive to re-

move this sludge from the sewers than from catch-basins. This experience was gained in days when the pavements of American streets were crude and little attention was paid to keeping them clean. The sewers themselves were not laid with that regard for self-cleansing velocities which is paid now. Under such conditions it was but natural that catch-basins should find more favor than they do at the present time. Durable pavements, more or less efficient street cleaning and sewers laid on self-cleansing grades, have reduced the need for such special

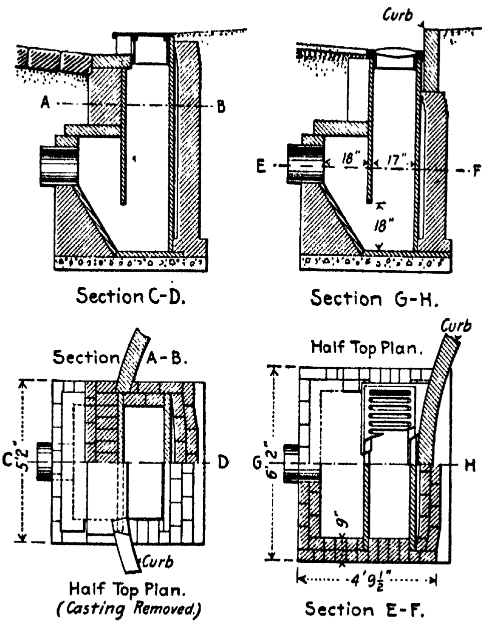


FIG. 199.—Standard inlets, Philadelphia.

structures to a few situations. The following quotations show the trend of opinion at the present time.

"We are also of the opinion that the inlets should not be provided with catch-basins to retain the filth or whatever may be washed into them. The object of such basins is to intercept heavy matter and periodically cart it away, instead of allowing it to reach the drains and there to deposit. Catch-basins, even after the sewage flow no longer exists in the gutters, are still apt to get foul because of the organic matter washed from the street. Such foulness is less offensive in the drains then in the catch-basins which

are situated at the side walk and where it is much more likely to be observed. Also, it is found impracticable to intercept all matter in the catch-basins which would deposit in the drains after they reached the flat grade in the lower part of your city. The cleaning of the drains would, therefore, be necessary in any event, and the additional amount of filth that would otherwise be intercepted by the catch-basins, will not cost much more to remove." (Report by Rudolph Hering and Samuel M. Gray on Sewerage and Drainage of Baltimore, 1896.)

"Theoretically desirable, catch-basins are, in reality, among the most useless devices employed for the removal of solid material from sewage. They are generally ineffective because they are not cleaned with sufficient frequency to enable them to serve as traps. It seems impracticable to keep them clean. To maintain catch-basins in serviceable condition requires much hand labor, and this is costly. The work is usually carried on to the annoyance of pedestrians and householders. Some sewerage systems are without catch-basins and their elimination, as a general procedure, is much to be desired." (Report, Metropolitan Sewerage Commission, New York, 1914.)

"That the sewers built by the Commission might become at once effective in providing for the disposal of storm water and thus fully useful at as early a date as possible, the Commission has built 225 storm-water inlets, of which some have been in the form of catch-basins. Careful consideration was given to the desirability of building inlets rather than catch-basins, as had been the city's custom for many years. It was felt, however, that in this climate it was unwise to provide pools of water in which mosquitoes could breed, as in the case where catch-basins are built, and further that under existing conditions the catch-basins, for the detention of detritus, were not necessary in most cases. It was also found that it was already the practice of the Board of Public Works to build inlets instead of catch-basins. The inlets as built have been untrapped and the experience thus far indicates that this type of inlet has given satisfaction." (Report to Commissioners of Sewerage of Louisville, by J. B. F. Breed and Harrison P. Eddy, 1913)

"In rural districts the gully retainers are often allowed to stand full of grit for months together, and any such detritus brought down by the rain thus runs straight into the sewers. If the retainers are not going to be emptied after each heavy fall of rain they might as well be omitted, as they are serving no good purpose, and may even cause considerable odor when they are allowed to stand full for long periods. In other places the gullies may only have to take water flowing on large paved areas where no mineral matter of any importance can reach them. In such positions the retainer merely serves to retain soft matter which would be better in the sewers. When we remember that a velocity of flow equal to 3.3 ft. per second will carry pebbles 1¼ in. in diameter along a sewer, and that a flow of 0.7 ft. per second will remove coarse sand, and that a flow of 0.5 ft. per second will remove fine sand, allowing every margin of safety, it seems that there can be very little object in taking so much trouble to exclude the washings of such paved roads. The author does not wish it to be understood that he thinks that retainers and traps are generally unnecessary,

but he considers that there are very many cases in which the traps and retainers might be omitted with advantage, and in which the comparatively fine grid might be omitted in favor of a larger opening." (H. S. Watson, "Sewerage Systems.")

"For these reasons the universal use of catch-basins is, in the author's opinion, not to be advised, but rather the inlet should be so designed that all materials shall at once reach the sewer. The inlet connection he would also make without a trap, that it may assist in the ventilation of the sewer; and if the sewer and its appurtenances are properly designed, constructed and maintained there will be very few instances where any odor can be detected at the inlet.

"The catch-basins should be cleaned after every rainfall. There is danger of putrefaction and objectionable odors from these, if this is not done within two or three days after each rain, but it is almost impracticable in large cities where there are one or two on every corner, without the use of an enormous number of men and carts, since each cart with three men will clean but 5 to 15 catch-basins a day. As an example of what is usually done in this line, a large city in New England, which is considered to have an excellent department of public works, during the whole of one year cleaned its 1100 catch-basins an average of 1.84 times each. It seems almost impossible that these catch-basins could hold the heavier matter washed from the streets during six or seven months, or if so the small amount contributed by each storm would have done little harm in the sewer, and the inference is that a large part of this was not held, but was washed into the sewer; also that the catch-basins were in an unsanitary condition a large part of the time. When so treated they might better be replaced with plain inlets." (A. Prescott Folwell, "Sewerage.")

In cities having smooth pavements and good sewerage systems there has been a tendency of late to look with favor upon the cleaning of street surfaces by flushing. In fact, a number of wagons have been specially designed for the purpose of forcing water under considerable pressure over the street surfaces, thereby causing the same general effect that is produced by the weak hose streams used in some European cities for the purpose. While a catch-basin does not actually prevent street cleaning by flushing, it is manifest that it is ridiculous to flush dirt into these basins and then raise it from them at far greater expense than is needed to collect it from the street surface; where flushing is to be employed, therefore, catch-basins should be omitted.

They are certain to be used, even in well-managed cities, as receptacles for street refuse which should be gathered otherwise according to ordinance. This was well stated as follows, by the Metropolitan Sewerage Commission of New York in its report of 1910:

"The men of the street-cleaning department wash some of the paved streets in certain sections of the city, and during this operation much detritus is carried into the catch-basins. The custom of pushing street sweepings into the basins appears to be quite general; and, in fact, the basins seem to

be popularly considered proper receptacles for anything that will enter the openings, including snow in winter. The report of the Bureau of Sewers for 1907 states that 9674 basins were cleaned of snow. Although there is an ordinance against putting snow and street sweepings into the basins, the magistrates have invariably dismissed the cases when the street cleaners have been arrested on complaint of the Bureau of Sewers for violation of the ordinances."

While there is not the crying need for catch-basins at frequent intervals which was formerly believed to exist, they have their uses where it is probable that large quantities of grit will be washed to the inlet and, if this enters the sewers, it is likely to cause obstructions in them. If they are used they should be cleaned whenever necessity arises. Cleaning should not be neglected until stoppage and the attendant flooding occurs, nor should basins be cleaned where there is little accumulation in

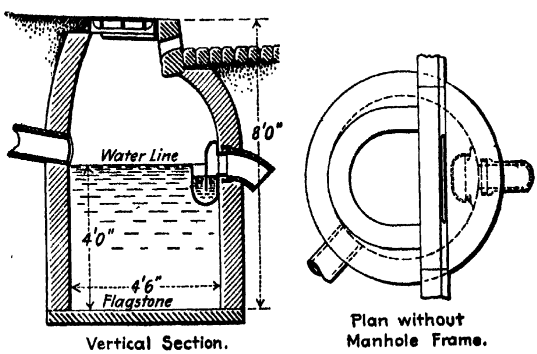


FIG. 200.—Standard catch-basin, Providence.

them unless in localities where the nature of the deposit is such as to create offensive odors which may escape from the basin and prove a source of annoyance to persons passing or living nearby. A basin may be put out of service automatically when it becomes filled. This is accomplished by the old-fashioned basin shown in Fig. 200, which represents a Providence structure. The feature of this catch-basin is the trap. As sediment collects in the catch-basin it reduces the space available for water above its top and below the water line established by the lip of the trap. Eventually there will be very little water capacity, and in summer, in prolonged dry weather, the water will evaporate to such an extent that odors may arise from the catch-basins. If no odors arise and the cleaning gang does not reach the basin in its regular routine, the sediment will gradually accumulate until it overflows the edge of the trap, blocking it. When this occurs the first heavy storm will give

undeniable evidence of the necessity of cleaning. In this way the trap serves a useful purpose by preventing the escape into the sewer of large quantities of silt which might form deposits. Another advantage of this basin, due to its trap, is that the water which accumulates in it can be bailed by the cleaning gang into the trap and thus delivered directly into the sewer, instead of being lifted to the top and thrown over the street. The great disadvantage of the trap is its liability to freeze in cold weather, although it should not be forgotten that the air inside the sewers, which will come up to the sewer inlet, will be somewhat warmer than the outdoor atmosphere, and the sheltered position of the trap also has some effect in reducing the danger of this nature. Where basins are connected to storm drains there will be much greater opportunity for the freezing of traps. Like all attempts to use traps on catch-basins or inlets, the

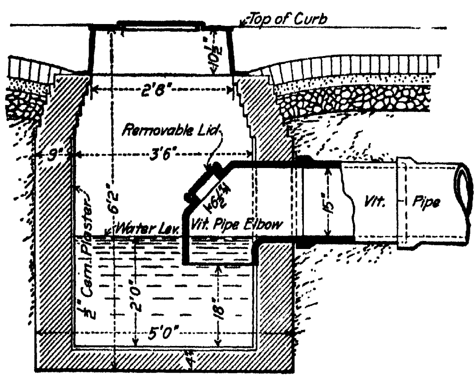


FIG. 201.—Standard catch-basin, Columbus.

permanence of the water seal is very questionable. It will evaporate during prolonged dry weather, and it is idle to expect that a sewer department will keep all traps filled by means of a hose during such seasons.

The type of catch-basins used in Columbus, Ohio, for many years, is shown in Fig. 201. It has two drawbacks, both due to the use of vitrified pipe for the elbow. It is difficult to believe that such vitrified elbows will withstand the hard knocks given to them during the operation of cleaning basins. This is rough work done as expeditiously as possible, and everything within a catch-basin should be designed to withstand hard usage. A second drawback to the basins for use in northern latitudes, is the possibility that ice will damage the elbow. The standard Newark catch-basin, Fig. 202, is typical of the form in

which the trap is in the wall of the basin. The standard catch-basin of the Borough of Manhattan, which is practically the same type as the standard of the Borough of the Bronx, is shown in Fig. 203. The illustration shows an inlet placed at the angle of a street corner and gives the dimensions of the structure so clearly that it is unnecessary to describe its details. The hood which forms the trap is hung from a plate and its main object is to prevent the entrance of refuse into the pipe running to the sewer, something which might occur if no hood was placed over its end.

One of the most unique catch-basins in the United States is probably connected with the Fourth Street sewer in Grand Rapids, Mich. It is at the bottom of a steep valley which leads down the side of an abrupt hill, a location which results in large quantities of sand and gravel, with occasional boulders, being washed into the inlet during every heavy storm. L. W. Anderson, while city engineer, carried on a number of experiments to determine some method of intercepting this sand and gravel and finally patterned his catch-basin after the separators used in manufacturing processes to re-

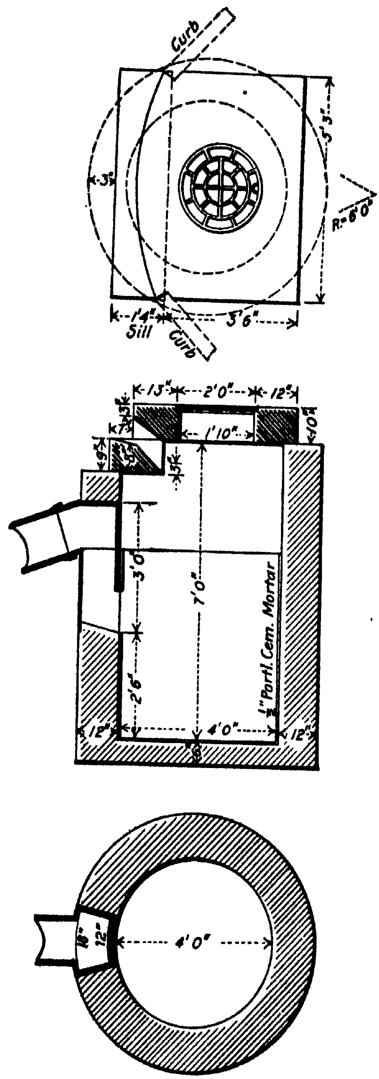


FIG. 202.—Standard catch-basin, Newark.

move shavings and dust from currents of air. The basin is an hexagonal reinforced concrete box 17-1/2 ft. between the parallel sides, in plan, and 11-1/2 ft. high, and is covered with a tight top as shown in Fig. 205, from *Eng. Record*, Oct. 24, 1908. At the center of the basin is a 2.5 × 3 ft. rectangular well which connects with the sewer. The top of this well is 2½ ft. below the under surface of the cover of the basin. Around the well is a nearly horizontal reinforced concrete slab, which extends to within 6 in. of the walls of the basin on all sides except the one at which water enters; on that side it is carried out to join with the wall of the basin. The upper side of this slab is 12 in. below the top of the well, and the slab is pitched 3 in. in all directions.

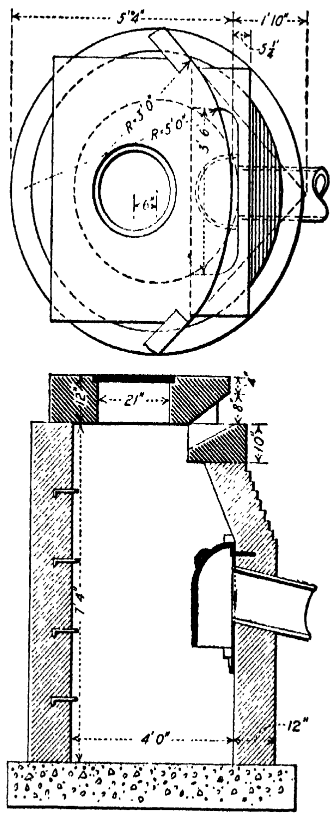


Fig. 203.—Standard catch-basin, Borough of Manhattan.

A 2 × 6.5 ft. inlet opening is made horizontally in the wall of the basin to which the slab around the well is joined. The top of this opening is 1 ft. below the lower surface of that slab. Directly in front of it a heavy protection is laid against the adjacent side of the wall, to receive the force of the water entering the basin. This protection is of the same width as the side of the basin, and consists of a mass of coarse gravel held in place by brick walls at both ends, and on the side toward the inlet by a 4-in. slab of reinforced concrete inclined at an angle of 60 deg. This inclined slab is covered with a course of paving brick laid in cement mortar, on which the entering water impinges directly. The interior of the protection consists of very coarse stone and the side brickwork is laid

with openings to permit the ordinary flow to escape through an opening in the base of the well.

The arrangement of the basin is such that the force of the water is broken as it enters by being directed against the inclined surface of the protection, the latter diverting the stream to the right and the left, and also vertically into the chamber of the basin under the slab around the central well. The only means for water to get from this chamber to the sewer is by passing up through a 6 in. opening between the walls of the basin and the slab around the well; and thence up over a 12-in. curb at

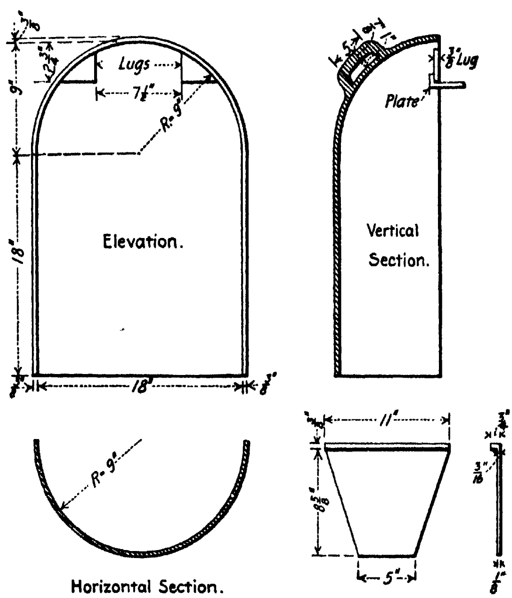


FIG. 204.—Hood for Manhattan catch-basin.

the well so that it drops into the sewer. The velocity of the water is thus greatly reduced at once, and the current is required to change its direction of flow at several points, so that all boulders, gravel and most of the sand are deposited in the chamber under the slab. Owing to the necessary upward flow of water, to pass out of the chamber through the 6-in. slot around the edge of the slab, only fine sand and particles of clay are carried above the latter. Most of these fine materials are also deposited on top of the slab, owing to the 12-in. curb which the water is required to surmount before reaching the well.

After each heavy storm it is generally necessary to remove several wagon-loads of material from the basin, according to the article in *Eng. Record*, published about a year after the structure was placed in service. This cleaning is readily done, it is stated, through two 3-ft.

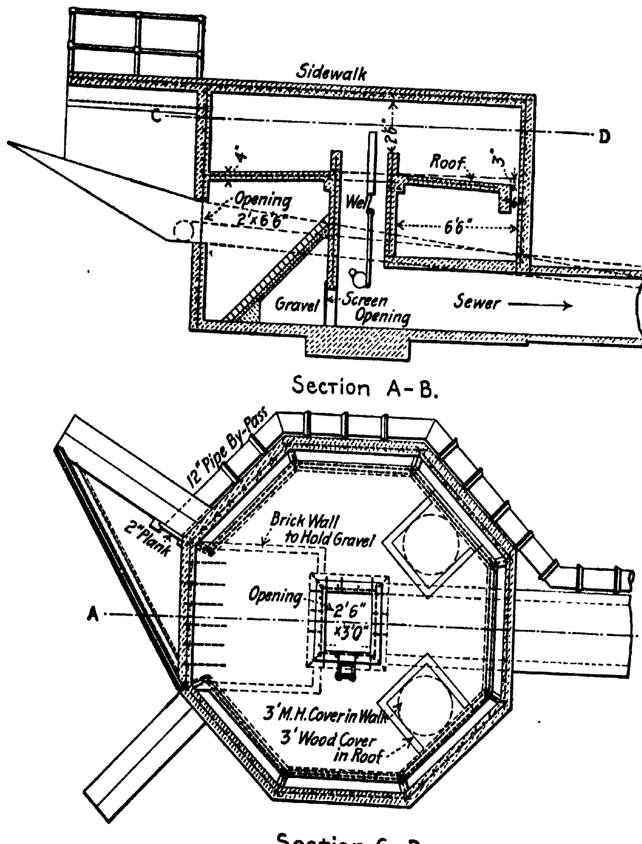
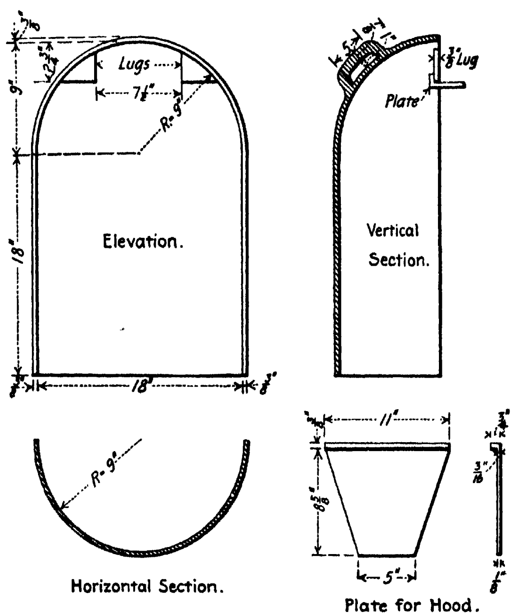


FIG. 205.—Catch-basin at Grand Rapids.

circular manholes in the cover. Directly under each of these is a 3-ft. opening in the slab forming the roof of the lower chamber of the basin, which openings are normally closed by covers held in place by eye bolts. Through these manholes it is possible to hoist the materials out readily,

with openings to permit the ordinary flow to escape through an opening in the base of the well.

The arrangement of the basin is such that the force of the water is broken as it enters by being directed against the inclined surface of the protection, the latter diverting the stream to the right and the left, and also vertically into the chamber of the basin under the slab around the central well. The only means for water to get from this chamber to the sewer is by passing up through a 6 in. opening between the walls of the basin and the slab around the well; and thence up over a 12-in. curb at



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much the same type except that the head is heavier and there is no entrance for water around the rim of the head. The D-pattern frame and grate were long used in Boston, but recently a rectangular frame has been adopted. There seems to be a general tendency toward these rectangular frames, of the general type indicated by the picture of the Merrimac frame, Fig. 206, and that of the standard Philadelphia inlet head, Fig. 207. They have two decided advantages over types having curves. The first is that it is practicable to keep the pavement of the gutter in better condition with a square than a curved casting for it to rest against. The second advantage is that the grate can be made as strong as desired without much difficulty and still have a large area

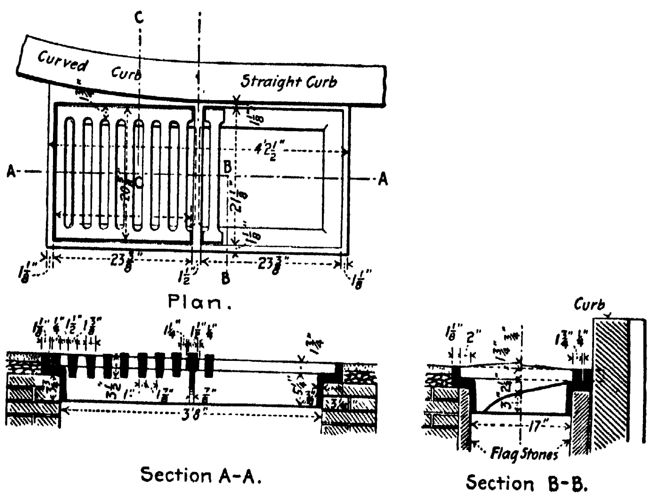

FIG. 207.—Standard inlet head, Philadelphia.

available for the passage of storm water into the inlet. The Borough of the Bronx adopted in 1913 a cast-iron inlet head shown in Fig. 208, which has a curb opening as well as the gutter drain. Whatever type is adopted should afford an opportunity for securely bedding the frame upon the masonry of the catch-basin or inlet, for otherwise it will become loosened speedily and in rocking under passing vehicles it will destroy the pavement about it. The North Berwick catch-basin frame is made for both 18 and 24-in. inlets, the Concord grates are made for 6 to 24-in. inlets, the Merrimac catch-basin frame is 24 in. square, measured on the cover, and the D-frame has a grate 24 in. wide and 26 in. long. In some cases the cover is in two pieces.

The material from which the frames and covers are made is rarely definitely specified. If anything is said about it, other than that it

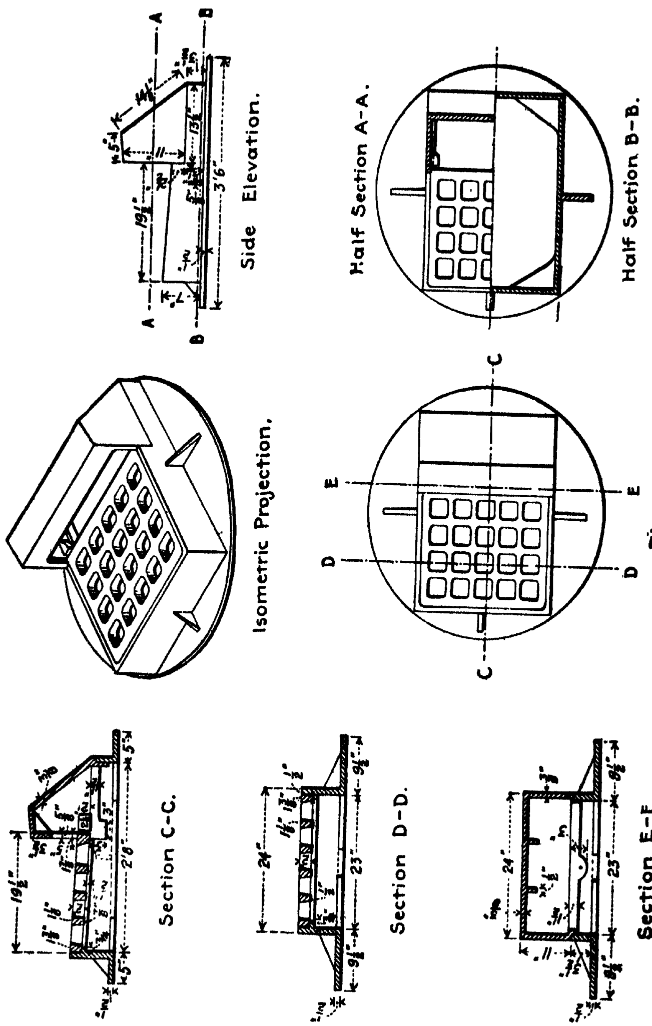

FIG. 208.—Inlet head of Borough of the Bronx.

must be cast-iron, the requirements are rarely more definite than that it must be of good quality and make castings strong, tough and of even

grade, soft enough to permit satisfactory drilling and cutting. It is not unusual to note a requirement that the metal shall be made without any admixture of cinder-iron or other inferior metal, and shall be remelted in the cupola or air furnace. The physical test usually required is that for the metal entering into cast-iron pipe larger than 12 in. As a matter of fact, it is not likely that the metal of these castings is ever tested or that there is any inspection of them at the foundry. Certain foundries have become known as furnishing good catch-basin castings and when they sublet such work to other foundries they hold up the quality of the product in order to protect their own reputation. There is a danger, however, in such loose specifications, particularly when a city calls for a large number of castings during a time of business depression. A foundry in a territory not ordinarily serving the city may conclude that it can manufacture poor castings which will meet the specifications well enough to make their acceptance legally necessary, and it can afford to send out such poor work, because it will probably never do business with the city again, owing to the freight rates against it. Such a situation has actually arisen and was met by the refusal of the mayor and superintendent of public works to award the contract for the castings to the lowest bidder, a decision which at one time seemed likely to bring them considerable newspaper notoriety of a most unpleasant nature. This danger can be avoided by requiring the castings to meet the standard specifications for gray-iron castings of the American Society for Testing Materials. (See Volume II.) The only additional requirements which are needed to make them apply to catch-basin castings are clauses related to the coating of the castings and similar minor details. The coating employed is usually an asphaltum, coal-tar or graphite paint.

MANHOLES

Although manholes are now among the most familiar features of a sewerage system they were not used extensively until some time after many large sewers had been constructed. They were introduced to facilitate the removal of grit and silt which had collected on the inverts of sewers having a low velocity of flow. Before that time, when a sewer became so badly clogged that it had to be cleaned, it was customary to dig down to the sewer, break through its walls, remove the obstruction and then close in the sewer again, ready to cause the same trouble at a later date. The opposition to the manholes seems to have been due to a fear of sewer air escaping from them, something which is not surprising in view of the contemporary accounts of the evil odors from defective drains. The engineers of the London parishes finally succeeded in obtaining authority to construct manholes, after they were

able to prove that it was much cheaper to remove the grit from sewers through them than to break a hole in a sewer each time it had to be cleaned. It was not until later, however, that the value of manholes on small sewers became recognized, and the principle became established that there should be no change of grade or alignment in a sewer between points of access to it, unless the sewer was large enough to enable a man to pass through it readily. There is one modification of this rule which has been permitted to some extent in the last 50 years, consisting of the use of a lamphole at changes in grade and more rarely at changes in alignment. Some engineers omit a manhole when it is closer than 200 ft. each way from other manholes, and substitute a lamphole. Such practice has never been general, and the use of lampholes in any situation is not regarded with favor by most engineers.

After the general acceptance of the principle that manholes should be placed at changes in line and grade in small sewers, there was a tendency for a time to go to the opposite extreme and put them in at too frequent intervals. This is objectionable because of the unnecessary cost and the inevitable injury to pavements caused by the presence of manhole frames in the roadway.

The earlier manholes were large structures, generally consisting of a flight of steps leading down to the sewer from the sidewalk or the roadway near the curb, and entering the side of the sewer. This position was chosen because it was believed that the refuse taken from the sewers would cause less obstruction to travel if removed at the side of the roadway than along its axis. This was more important with the old sewers on very low grades, from which large quantities of grit were removed, than it is to-day, when the sewers are on better grades and the amount of grit entering them is probably less than it was 50 years ago. The experience with these side-entrance manholes was quite unsatisfactory, for during every period of storm flow the side entrance and the lower steps of the manhole became covered with filth, which remained there when the sewage level dropped to its normal dry-weather stage, resulting in decidedly unpleasant conditions when the weather was warm. It was found that such manholes could not be kept clean so well as those having a plain shaft, with the sewage confined in channels in its bottom. Furthermore, the actual obstruction caused on the surface of the street by the men engaged in removing material from a manhole was found to be insignificant in most cases.

The great majority of manholes are constructed of brick, although under some conditions concrete may possibly be used to advantage, particularly where a large number are to be built, so that standard forms may be utilized, or where the manholes are very deep, requiring considerable masonry. The expense of procuring forms and the delay which their preparation frequently entails, the difficulty of placing them and of

placing the steps in the concrete, and the small quantity of concrete which is used, usually make it more economical to employ brick upon ordinary manhole construction.

The manholes of small sewers are usually made about 4 ft. in diameter when of circular cross-section, or about 3 × 4 ft. when an oval cross-section is employed. The same size is usually maintained for all sewers except when special conditions may require manholes of larger size, as when gaging devices must be used at the bottom of the manhole, or it is desired to have considerable storage capacity in the manhole chamber to enable this to be used to flush a long line of pipe on a flat grade. Brick manholes are usually built of 8-in. brickwork down to a depth of 12 to 20 ft., although until recently the manholes upon the Cincinnati sewers have been built of a single ring of brick, and possibly this practice has been followed in some other places. Below the depth stated, 12 in. of brickwork is used as a rule. The sides are carried up vertically to within 3 or 4 ft. of the top and the upper part is corbelled in or laid in the form of a dome or reverse curve. These three types of construction are shown in Fig. 209.

In wet and yielding material, care must be taken that the unit pressures on the foundation of the manhole and the foundation of the sewer

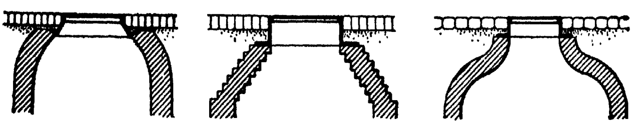

FIG. 209.—Types of manhole tops.

are approximately uniform, for otherwise there is danger of a settlement of the manhole, which will break the connection with the sewer. If the pressures are not normally the same, a spread foundation may be built to reduce the unit load imposed by the bottom of the manhole. When manholes are built in sewers having a diameter approximately that of the manhole, the walls of the latter are started directly from the side walls of the sewer, as shown in Fig. 218. In the case of brick sewers a ring of brickwork surrounding the opening should be laid with joints approximately radial to the center of the manhole, so as to form a cylinder to take the thrust of the sewer arch at the point where it is cut away. As a general proposition, in fact, care should be devoted to the junction of all shafts with a sewer, for the pressure of the surrounding earth is likely to bring unexpected strains on such junctions, which cannot be calculated with any degree of accuracy. The stability of the structure can be assured by avoiding details which will give an opportunity for the backfill in settling to impose heavy loads on branches or

lines of junction where it is difficult to provide extra strength without high additional cost.

Where the sewer is much larger than the diameter of the manhole, the outside of the latter is usually tangent to one side of the sewer, for otherwise it will be difficult to enter the sewer and a special ladder will be required to reach the invert. When the sewer is very large, the whole manhole may rest on the steep side of the arch, and care must be taken to bond it with the latter carefully. This may be done by having some of the brick in the outer ring of the sewer arch and under the position of the manhole walls project out half their length to act as headers. A horizontal tread may then be built up with these brick as a base, and the manhole wall started from it. Occasionally, on very large sewers, the manholes are built entirely apart from the sewer proper and have a shaft leading into it, as shown in Fig. 210.

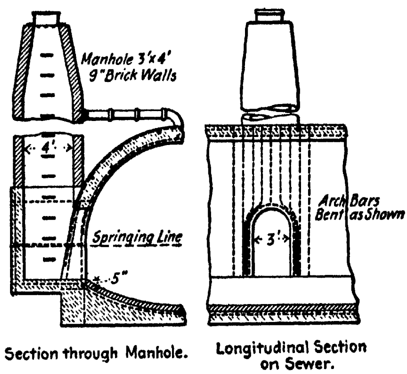

FIG. 210.—Manhole on large St. Louis sewer.

The four manhole bottoms shown in Fig. 211 illustrate somewhat different types of design. The Memphis and Seattle bottoms have flat lower surfaces while the Concord and Syracuse bottoms have lower surfaces curved to correspond with the channels through them. Which type of base is best adapted for the soil at any site can only be ascertained by examination; the saving in material in the second type may be counterbalanced by an increased unit cost. While the base of each manhole illustrated was constructed of concrete, as a matter of fact a good sewer mason can lay up brickwork to form practically any channel that may be desired, and can carry the work on very expeditiously, if he is so minded.

The channels in the bottoms of the Memphis and Concord manholes are not provided with high walls, the Concord channel being nearly semi-

circular and the Memphis channel hardly more than that. On the contrary, the channels of the Seattle and Syracuse manholes have such high walls that they will carry all the sewage until the sewers become surcharged. It is now considered desirable to have the walls of the channel rise nearly to the crown of the sewer section, and then be stopped in a berm, which is given a slight pitch from the wall toward the channel. The standard manhole used in Newark, N. J., for many years, which is

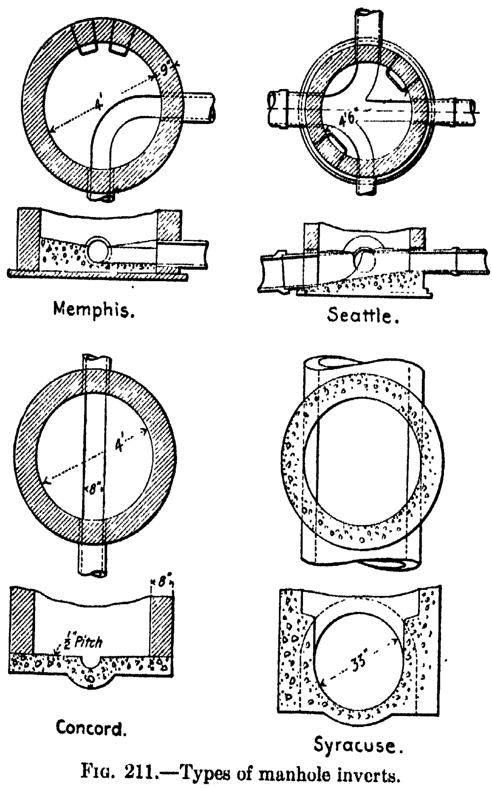

FIG. 211.—Types of manhole inverts.

shown in Fig. 212, illustrates this form of construction carried a little farther than is perhaps customary. The standard Philadelphia manhole bottom, Fig. 213, illustrates the method of giving a little extra velocity to the sewage leaving the branches, by providing a steep grade for the invert within the manhole.

Concrete manholes have been used in Syracuse on concrete intercept-

ing sewers. Two types have been employed. In the first type the manhole has a reinforced shell 6 in. thick, running up from the sewer to within

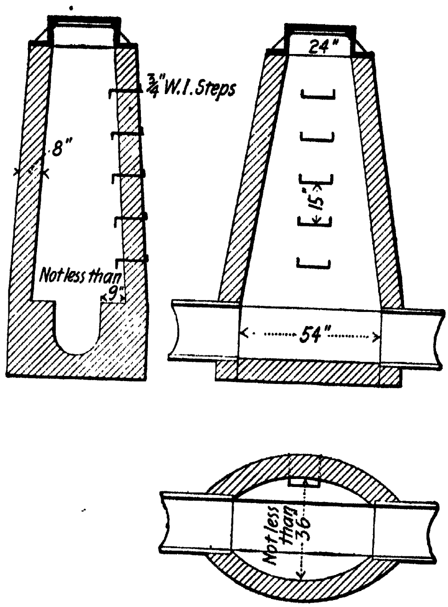

FIG. 212.—Standard manhole, Newark.

5 ft. of the ground surface, where a funnel-shaped top begins to corbel in. The reinforcement consists of 1/2-in. rods spaced 12 in. apart when

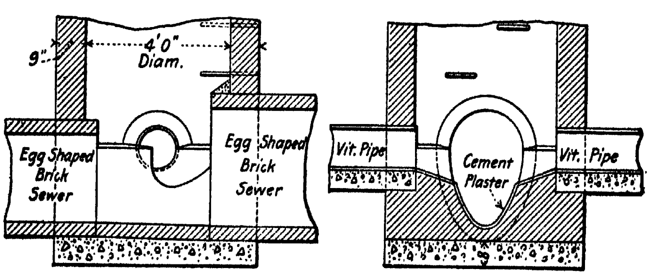

FIG. 213.—Standard manhole invert, Philadelphia.

horizontal, and 18 in. apart when vertical. The other type of concrete manhole is formed of reinforced concrete pipe placed on end. The sec-

tions are 4 ft. in diameter and 4 ft. long, and were constructed like the reinforced concrete sewer pipe used in the same city and described in Chapter X.

The manholes built on the Winnipeg sewerage system, of which Col.

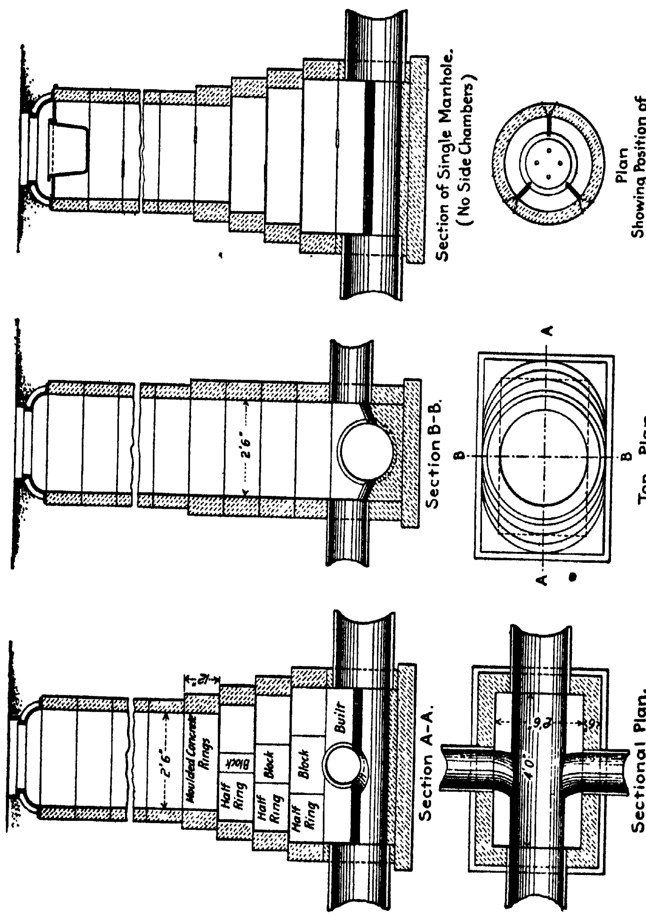

FIG. 214.—Standard concrete manhole, Winnipeg.

N. H. Ruttan, the city engineer, has been the designer since its inception, are constructed of concrete rings 30 in. in diameter, 4 in. thick and 12 in. high, except the bottom four rings. These are made 6 in. thick and are split in halves to permit concrete blocks 6, 12 and 18 in. wide, to be

inserted in the third, second, and first rings, measuring from the bottom, Fig. 214, so as to lengthen the lower portion of the shaft in the direction of the axis of the sewer and allow it to rest on the monolithic base in which the invert is formed. This base has an inside length of 4 ft. and a width of 30 in. and its walls are 6 in. thick.

Double manholes are sometimes used where the sewers and drains are so located as to make them convenient. The structure shown in Fig. 215 was used by the authors for such a purpose on the separate sewerage system of Hopedale, Mass. Each chamber is 5 × 4 ft. in plan and the dome has a depth of 4 ft. The walls are 9 in. thick.

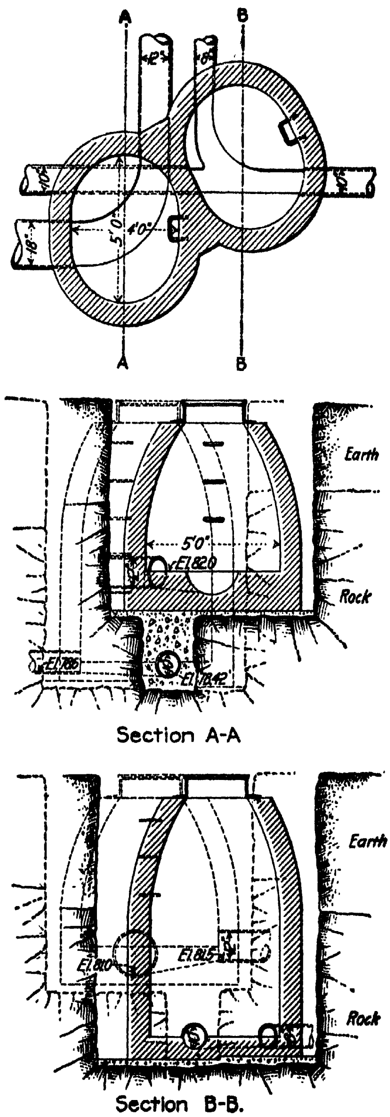

FIG. 215.—Double manhole for separate system.

Where underdrains are employed it is sometimes desired to afford access to them, and in such cases various expedients are employed. The most usual is to divert the underdrain a short distance to one side of the sewer, where it passes under the manhole, and to bring up a riser to the floor of the manhole, some-

what as shown in the illustration of the manhole at the head of the Woonsocket siphon, Fig. 249. Where an underdrain is dropped along with a sewer, as in the drop manhole shown in Fig. 219, some such provision for giving access to the lower end as is there illustrated, may be provided.

The Lovejoy combination manhole, quite largely used in Boston and its vicinity, is a patented structure shown in Fig. 216 and controlled by the Gibby Foundry Co., East Boston. The characteristic feature of the design is the storm drain, crossing the manhole above the sewer and provided with a large opening closed with a removable cover, which can be held so firmly in place that there will be no leakage at the joint, even when the drain is surcharged.

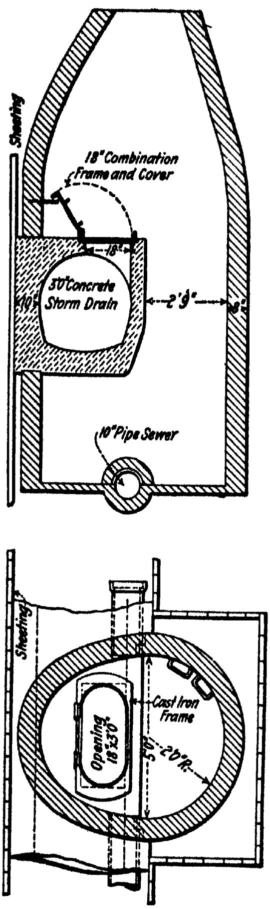

Fig. 216.—Lovejoy combination manhole (Patented).

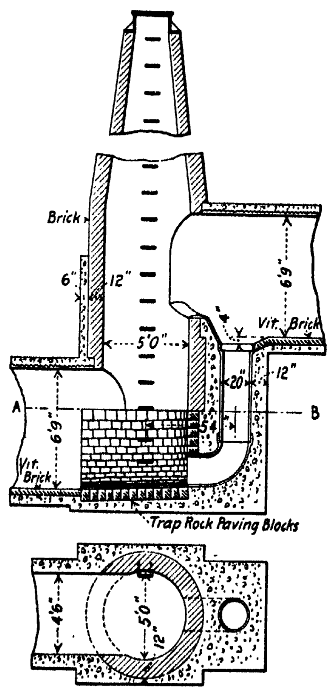

Fig. 217.—Drop manhole, Staten Island.

Drop Manholes.—The drop manhole, sometimes termed a "tumbling basin," has a mild historical interest as being the subject of patent intimidation and litigation which was an annoying feature of sewerage work in the Central States for a number of years. In 1892 a patent

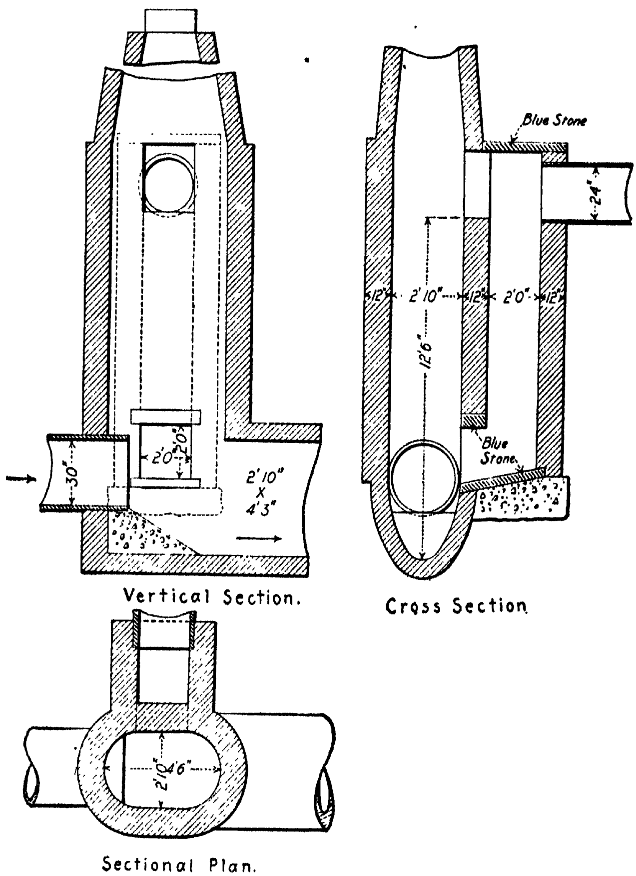

FIG. 218.—Drop manhole, Newark, N. J.

for the drop manhole was granted to James P. Bates, and assigned to Alexander Donahey, of Kirksville, Mo. Thereafter, whenever a city adopted plans for a sewerage system with drop manholes, it was likely to receive a notification of litigation for infringement of the Bates

patent unless a license fee, usually $10 per manhole, was paid. The sum demanded was so small that the city counsel usually advised its payment, although city engineers strongly fought against it in the courts. Finally the city of Centerville, Ia., decided to test the matter and refused to pay a license. Suit was brought, but on April 16, 1907, the U. S. District Court sitting at Keokuk, ruled, before the defense had introduced its testimony, that the drop manhole had no patentable features. That ended the matter.

The drop manhole shown in Fig. 217 was constructed on a 6 ft. 9 in. × 4 ft. 6 in. sewer on Staten Island. It has a 20-in. cast-iron drop imbedded in concrete, for the dry weather flow, and it will be observed that the general arrangement is such that even in times of heavy discharge the flow down this drop pipe probably serves to form a cushion at the bottom of the manhole, to receive the bulk of the storm-water flow. It may be added as a matter of interest that on one sewer on Staten Island there are 29 drop manholes in a length of 7883 ft. Fig. 218 shows a drop manhole built in Newark, N. J., which is rather unusual on account of its location at the head of a large oval brick sewer 4 ft. 3 in. high, into which two circular sewers discharge at different elevations. The drop manhole shown in Fig. 219, was constructed at Medford, Mass., under the direction of T. Howard Barnes. In an article in *Engineering Record,* Oct. 30, 1897, he stated that the sub-drain inspection hole had been found very convenient. Frequently it served at times of making connections with constructed work, as a well through which to lower the adjacent ground water. A still more unusual type of drop manhole and underdrain overflow is shown in Fig. 220. This was constructed on the sewerage system of Newton, Mass., from the designs of the late Albert F. Noyes. The drop takes place through a sheet-iron funnel and pipe. The bottom of each standard manhole on the two sewers shown in the plan has a central opening into the underdrain, with the channel divided and passing around the opening in twin inverts, a type of construction which was introduced in some other places by this engineer.

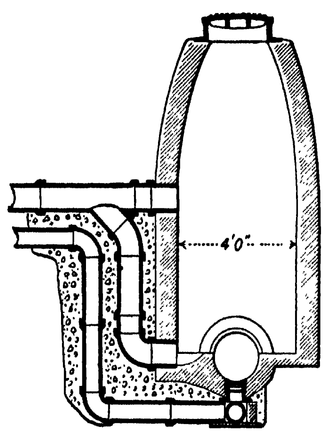

FIG. 219.—Double drop manhole, Medford, Mass.

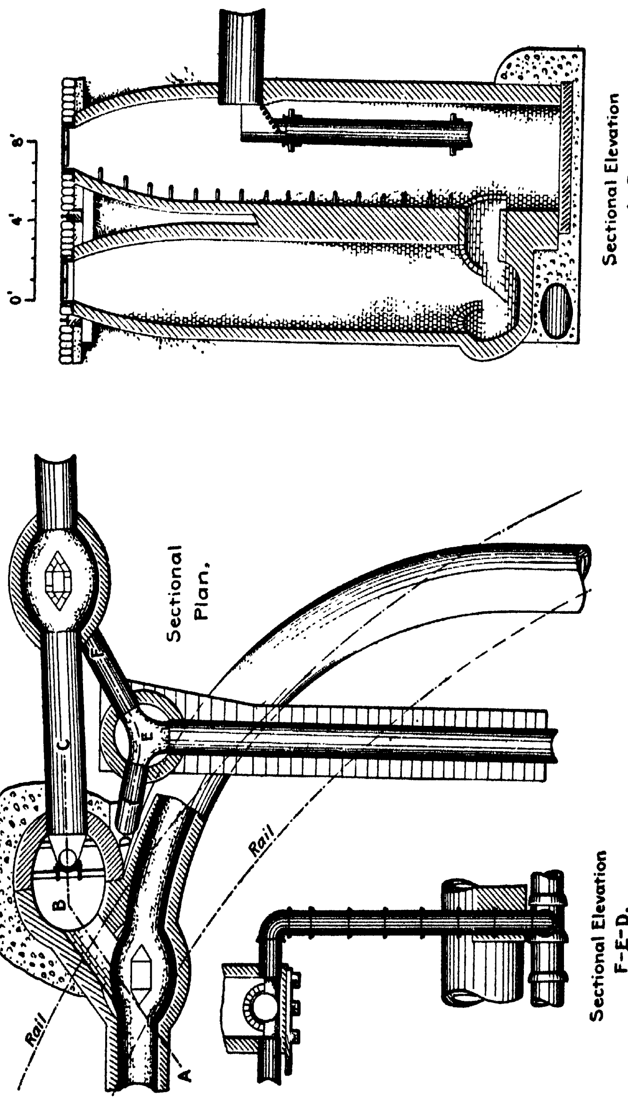

FIG. 220.—Drop manhole and underdrain overflow, Newton, Mass.

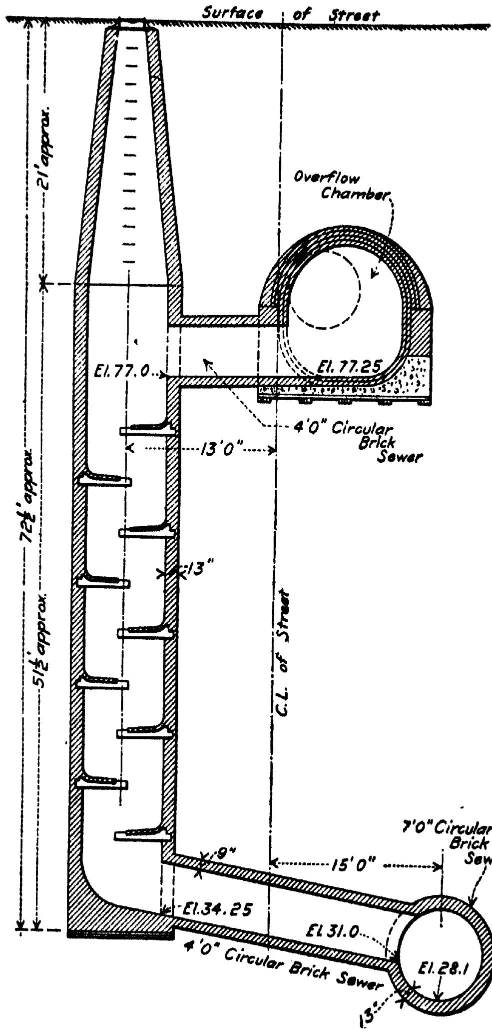

FIG. 221.—Wellhole, Morgan Run sewer, Cleveland.

Wellholes.—Deep manholes in which the sewage is dropped a considerable distance from one elevation to another are sometimes called drop manholes, although that name belongs to the type just described, and are more frequently termed "wellholes." Fig. 221 shows such a structure on the Morgan Run sewer in Cleveland, a city which has had considerable experience with these wellholes.

A wellhole 65¾ ft. deep from the surface of the ground to the bottom of the invert, Fig. 222, was built in 1893 in Petrie Street, Cleveland, where the roadway was carried on a very deep fill. In order to check the velocity of fall of the sewage the latter dropped at intervals of 5 ft. on stone flagging, having a thickness equal to that of two courses of brick, placed as shown in the illustration. The connection from the bottom of the manhole to the 6-ft. culvert, was of a flexible character, as indicated in the sketch, owing to the probability that there would be some settlement under the fill in the course of a few years. After this settlement had occurred it was proposed to calk the joints of the connection thoroughly from the inside. Whether this was done cannot be learned but the structure served its purpose satisfactorily for about 10 years, when it was abandoned on account of the reconstruction of the Petrie Street sewer.

Some very deep wellholes have been constructed at Minneapolis, in connection with the sewers built to discharge storm water into the Mississippi. The greater portion of the city served by these sewers is from 80 to 100 ft. above the river. Along the river bank is a drive and park which make it necessary to build the wellholes some distance from the river. The typical wellhole shown in Fig. 223, from *Eng. Record*, April 8, 1911, is 340 ft. from the outlet, for example. Where the drop is through hard limestone the section is not lined but given a funnel shape, which is advantageous in concentrating the sewage in the center of the lined portion of the wellhole. This latter has a lining of granolithic block in a backing of concrete, and the outlet sewer from it starts at an elevation which gives a deep sump in the bottom of the wellhole, forming a water cushion to prevent erosion of the lining by the falling waters.

On some of the tunnel sewers in the Borough of Brooklyn, there are manholes from 65 ft. to 83 ft. in depth, Fig. 224, into which sewers discharge at distances of 25 to 40 ft. above the invert of the main sewer below. Below these shaft manholes the invert is paved with granite blocks laid in Portland cement for a distance of as much as 30 ft. Furthermore, although the trunk sewer is in a tunnel at this place, an extra heavy bottom is constructed below the shaft and manhole for a length of about 14 ft.

The use of drop manholes and other special details to give a sudden drop in grade is not regarded with favor by some designers. For

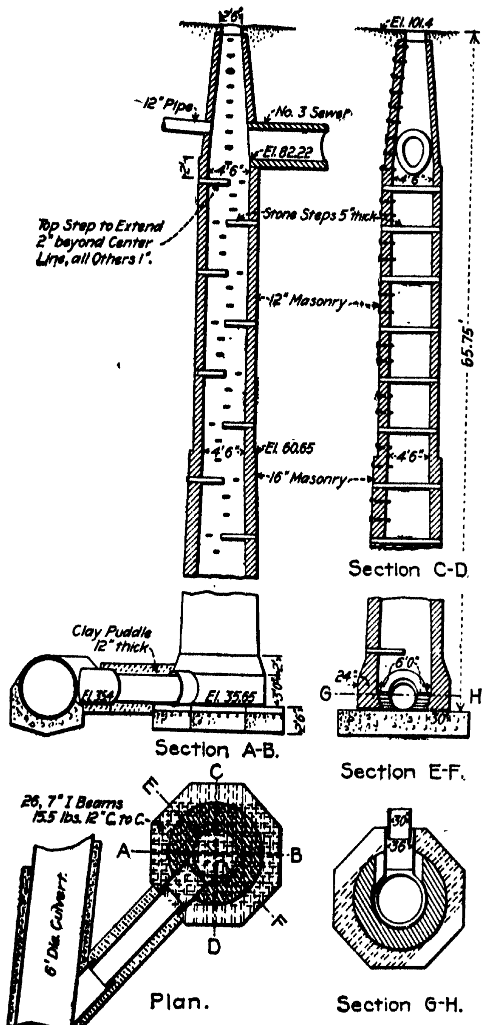

FIG. 222.—Wellhole, Petrie Street sewer, Cleveland.

example, W. W. Horner, of the St. Louis Sewer Department, stated in an article in *Engineering News*, Sept. 5, 1912, that "the tumbling basin introduces unknown factors into a sewer system, which we now think best to avoid, if possible. It is questionable whether the basin really acts to advantage under extreme conditions. Such construction is very expensive, for if the sewer is deep enough above the basin, it is too deep below, involving excessive excavation; also, if it is supposed to

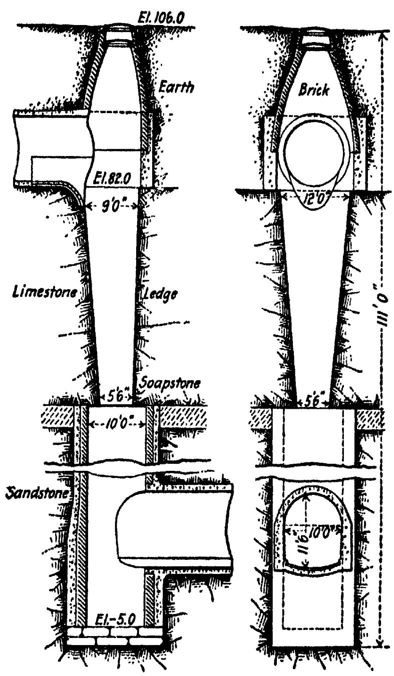

Fig. 223.—Wellhole, Minneapolis.

check the velocity, much larger sewers are required for the flat grade. The present practice (1912) is to design the sewers carefully at all points and to take advantage of all the natural fall, in order to decrease the size of the sewers; then to build them strong enough to take care of the resulting high velocities." Where sewers are built in deep rock cut, the high cost of excavation has frequently led in St. Louis to the adoption of a rectangular cross-section for the sewer. By making the sewer narrow and high the amount of excavation will be materially decreased,

but as the ratio of the height to the width increases, the section becomes less efficient from the hydraulic view-point, requiring a greater wetted area for the same capacity. A number of conditions must be fulfilled in such cases, and the best section can only be obtained by a number of trial calculations.

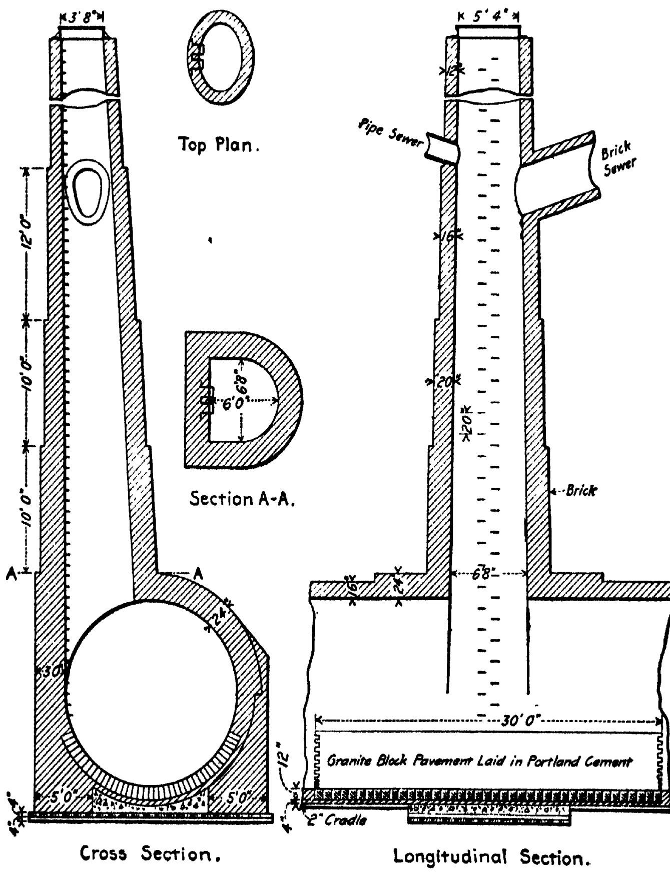

Fig. 224.—Wellhole, Borough of Brooklyn.

Flight Sewers.—A considerable fall must sometimes be provided in a sewer, and while a drop manhole or wellhole always affords a means of changing grade sharply, the lower sewer which leads from such a shaft

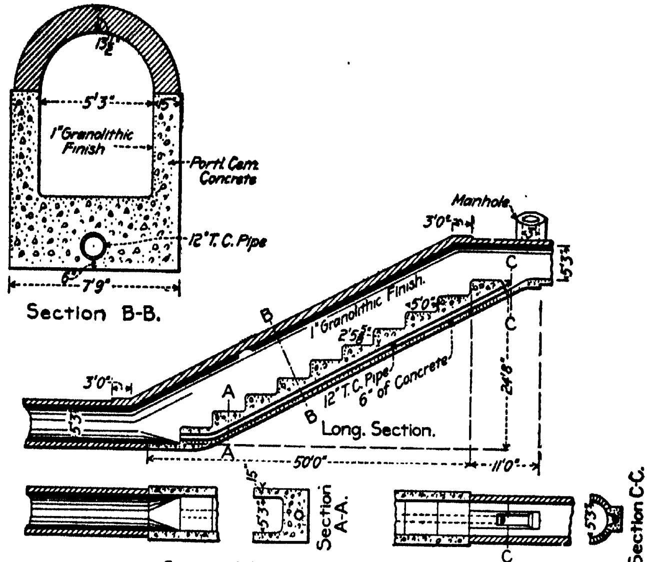

Fig. 225.—Flight sewer, Philadelphia

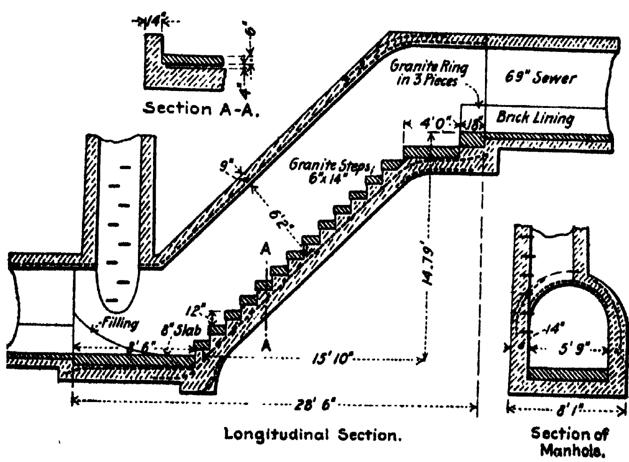

Fig. 226.—Flight sewer, Baltimore.

may be so deep that any prolongation of it should be avoided if a less expensive structure can be made to serve. The flight sewer, which gets its name from its resemblance to a flight of stairs, is occasionally used in such situations. It has a steep grade, but steps in the invert tend to check the velocity of the current; the resistance they offer probably diminishes with the depth of the sewage, and if the descent is long great care should be exercised to ensure massive, durable construction and freedom from obstruction to flow at the bottom of the flight, which may be seriously strained if the sewer should ever run full. Two examples of such a sewer are shown in Figs. 225 and 226, from *Engineering Record;* the first has a small circular channel within the concrete base to carry the dry-weather flow while the second has no such provision.

The flight sewer shown in Fig. 225 is a part of the Indian Run sewer in Philadelphia. The total length of this special section is 61 ft., and in that distance there is a drop of 24 ft. 8 in. The granolithic finish of this section was a mixture of one part cement, one part sand and one part granolithic grit. On the risers this mixture was placed against the face of the forms in advance of the bulk of the concrete filling. The layer was at least 1 in. thick in every place. After the forms were removed the face was at once brushed with a thin plaster of equal parts of sand and Portland cement.

A drop of 15 feet is made on one of the sewers of Baltimore by means of the flight sewer shown in Fig. 226. This sewer is near the high service reservoir and in this vicinity there is another flight sewer of 10 ft. drop.

Special Manholes.—Angle wells are occasionally used on large aqueducts under light to moderate pressure, where it is impracticable, except at excessive cost, to put in horizontal curves fitting the topography. In a 60-in. pipe line laid in 1912 by the Denver Union Water Co., for example, four of these wells were used (*Eng. Record*, Jan. 18, 1913), where the angles were 27° 8′, 33°, 35° 27′ and 20° 28′ respectively. They were 8 ft. in diameter, 10 ft. high and made of 3/8-in. steel, and were held down by eight 1-1/2-in. anchor bolts attached to angle fasteners riveted to the sides. In addition to providing change in direction, these wells were expected to act to some extent as sand-catchers, as they extend 2-1/2 ft. below the bottom of the pipe.

The gaging manhole shown in Fig. 227 was built at Liberty, N. Y., from the plans of Wise & Watson, of Passaic, in 1900. This manhole is provided with a triangular weir, which is rather unusual in that it is an equilateral triangle rather than one having a top width twice its center height. The latter shape has been adopted mainly as a result of experiments made by Prof. James Thomson and reported in the British Association Report, 1858, page 133, and of experiments by Prof. Dwight Porter at the Massachusetts Institute of Technology. At the present time there is need of some accurate investigation of the discharge of

water through triangular notches, for the uncertainty regarding the subject prevents their use in gaging small flows where their form would make them particularly applicable were there greater certainty regarding their results. Further information about this weir was given in Chapter IV.

Information regarding other forms of gaging manholes is given in Chapter IX, on gaging storm-water flow in sewers.

A manhole for an unusual purpose is illustrated in Fig. 228, from *Eng. Record*, Aug. 28, 1909, and it also is of interest in that it is one of the very few structures where life has been lost owing to the poisonous effect of sewer air. This structure is at the end of the Los Angeles sewer outfall where it discharges into a wood stave pipe that carries

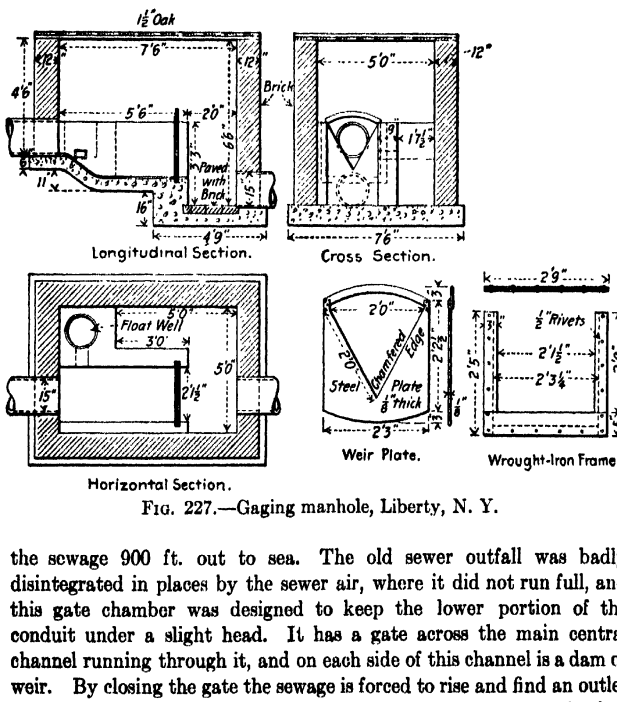

FIG. 227.—Gaging manhole, Liberty, N. Y.

the sewage 900 ft. out to sea. The old sewer outfall was badly disintegrated in places by the sewer air, where it did not run full, and this gate chamber was designed to keep the lower portion of the conduit under a slight head. It has a gate across the main central channel running through it, and on each side of this channel is a dam or weir. By closing the gate the sewage is forced to rise and find an outlet over the crests of the two weirs. One of the engineers of the city lost his life in 1909, in manipulating the hand wheel by which the gate was raised and lowered. With a companion he moved the gate a number of times, and the companion reported that whenever the gate was near its

seat the violent rush of sewage below the bottom of the gate gave off gases which caused extreme dizziness. They were several times forced to come to the surface and lie down; the engineer lost his life on one of these occasions. Instead of leaving the manhole he stood partly

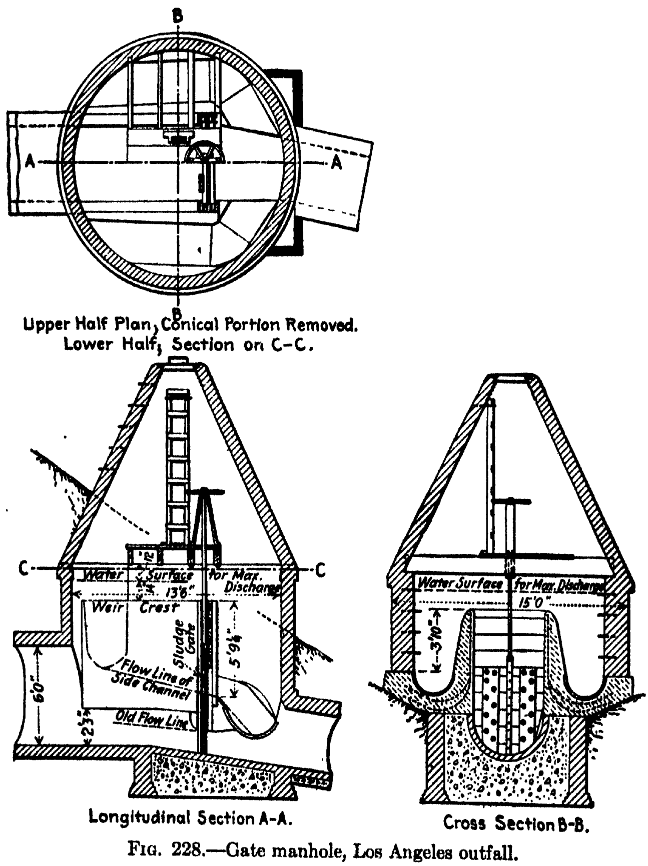

FIG. 228.—Gate manhole, Los Angeles outfall.

out of it, his arms resting on the manhole frame and his feet on the ladder. He was suddenly seen to drop, and when his companion hurried to the gate chamber his body could be seen resting on one of the steep side inverts, from which it slipped into the wood outfall; a few days later it was found floating in the water.

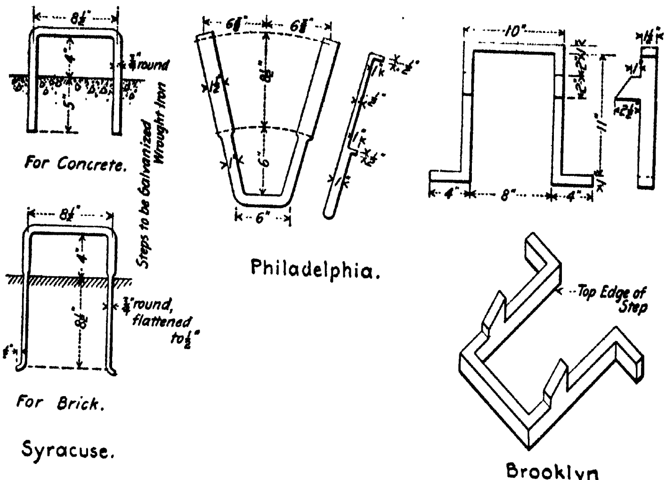

FIG. 229.—Types of step forgings for manholes.

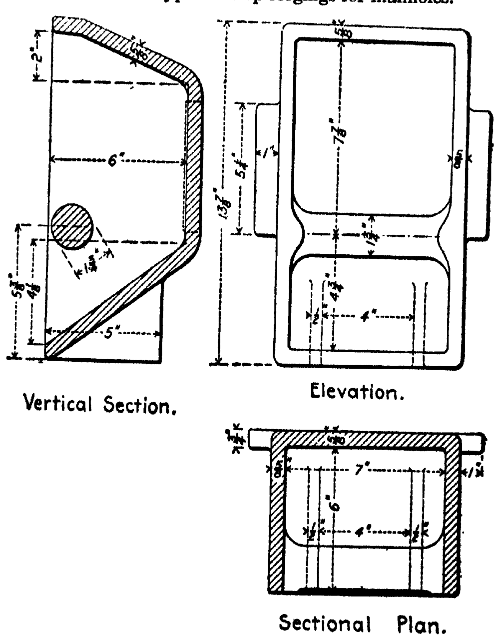

FIG. 230.—Cast-iron box step, Boston.

Manhole Steps.—In shallow manholes, steps are sometimes formed by leaving projecting bricks at the proper points, about 15 in. apart vertically. This is an old practice and while not approved by many engineers, has been in more common use than any other method of construction until quite recently. The steps are objectionable because they are sometimes slippery, when it is difficult to use them safely, and, moreover, they are subject to breaking.

The usual method of providing steps at the present time is to construct them of forgings, which are bedded in the brickwork or concrete. Three types of these steps are shown in Fig. 229. Sometimes steps are formed by straight rods inserted in the masonry in such a way as to form chords of the brickwork ring, with the center of the step at least 4 in. from the brickwork. The steps are usually placed from 12 to 18 in. apart vertically and somewhat staggered; a number of cities seem to be in favor of a vertical spacing of 15 in. The authors have found the step of the type marked "Syracuse" in Fig. 229, satisfactory, but experience with it indicates that the blacksmiths who forge the steps must be cautioned to follow the dimensions accurately, for otherwise there will be trouble in fitting the steps into the joints of the brickwork. Fig. 230 shows a cast-iron step used in Boston where the shaft must be kept free from any projections from the wall.

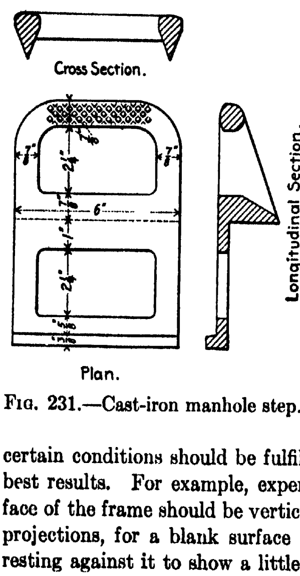

Fig. 231.—Cast-iron manhole step.

Manhole Frames and Covers.—There is just as great a variety in manhole frames and covers as there is in the castings for catch-basins and street inlets. It is gradually becoming evident, however, that certain conditions should be fulfilled in the design in order to get the best results. For example, experience now indicates that the outside face of the frame should be vertical from bottom to top and be without projections, for a blank surface of this nature enables the pavement resting against it to show a little better resistance to wear than is the case where there are projections at the top of the frame or the latter has a broken surface. Again, the practice of making the covers rather deep and having a pocket in their top, in which asphalt or wood block is placed, once much favored, is now (1913) regarded with much less favor by city engineers, who are recommending instead a cast iron cover with the surface broken by a shallow pattern of some sort which will

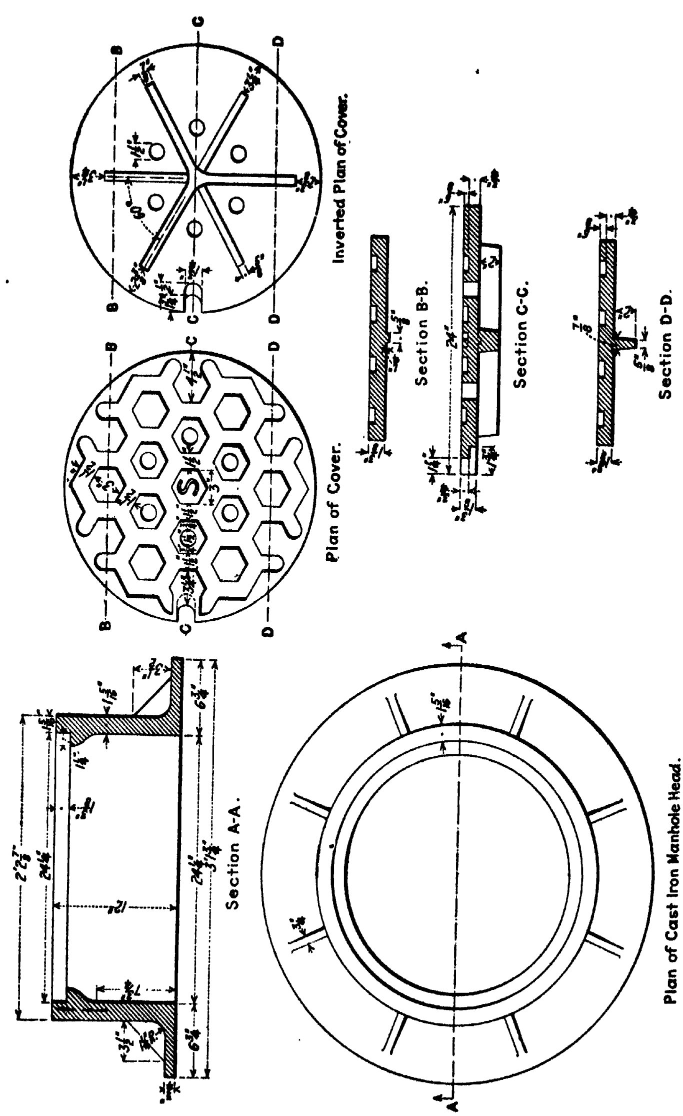

FIG. 232.—Standard manhole head and cover, Borough of Manhattan.

give resistance to slipping when horses step on the castings. The present tendency is undoubtedly toward somewhat heavier and simpler frames and covers than were used at the beginning of this century.[1] This is well shown in Fig. 232, illustrating the standard manhole head and cover Manhattan, adopted in 1912. The head has a minimum weight of 475 lb. and the cover 135 lb. The cover is raised by inserting the end of a pick or bar in the recess, *C*. The standard cover and frame used in Philadelphia are shown in Fig. 233. This has an opening of practically the same diameter as the Manhattan frame, but weighs very much less. With a ventilating cover, in which the rectangles marked *V* are left open, the weight of the frame and cover, together, is 340 lb.; with a closed cover, the weight is 365 lb. This cover is raised by means of the loose links, which are easily lifted from the grooves in which they rest.

With this frame is shown the wrought-iron bucket used below the cover when the latter is provided with openings for ventilation. The bucket is constructed of $\frac{1}{16}$-in. galvanized iron and has three lugs by which it is held in place in recesses in the ring of the frame supporting the cover. The bucket can be lifted by means of the two bent handles. The bottom contains a number of half-inch holes, to allow water entering through the ventilated cover to drop into the sewer. This method of allowing water to escape has been found in some places to permit the escape also of most of the dirt which falls into the bucket, for it is in the form of very fine powder in such cases, and is not long in finding its way down the manhole into the sewer. Where the refuse to be caught is coarser than dust, the perforations in the bucket are less criticized by engineers who have tried them.

The use of ventilating covers is considered necessary by some engineers when a network of sewers is first constructed and there are few house connections with it to afford ventilation. After the system has been in service for some time, there seems to be a general tendency to use closed covers on a considerable portion of the manholes and employ the open covers only where the need for them is evident. The openings in the covers are often closed with oak plugs, but the authors have found that the best way is to have a blacksmith plug them with rivets. Attempts to fill them with cement or an asphaltic mixture are not successful for any length of time, as a rule. In some cases the manhole covers are provided with grooves for a slide below the perforated portion, and when the cover is to be closed the slide is inserted in place, closing

[1] In a letter from George W. Tillson, Consulting Eng. to the Borough of Brooklyn, the desire of the street department officials was stated to be a cover "which will present as little obstruction to traffic as possible and interfere the least with the construction of the pavement. A head without a line parallel to traffic is preferable, as with square heads which have a line parallel to the line of traffic, ruts are liable to form."

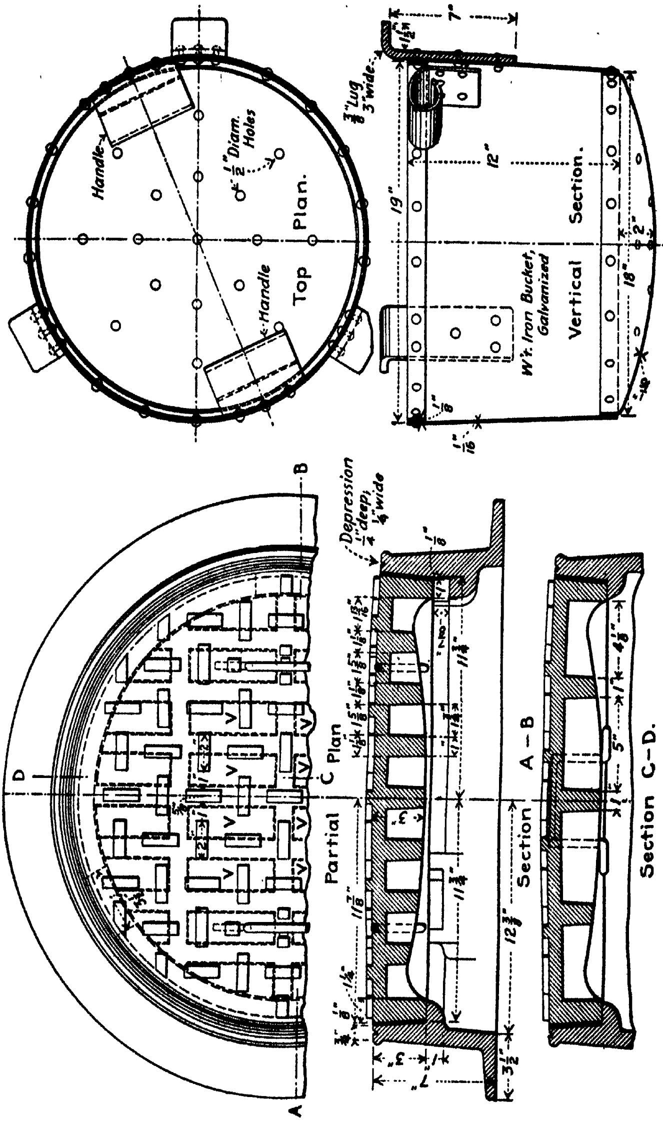

FIG. 233.—Standard manhole head and cover, Philadelphia.

the bottom of the holes, which are then filled with grout or an equivalent material.

Just how much rain-water enters through the perforations in these covers is difficult to estimate, because of the fact that the ground-water level and the leakage into the sewers generally rise at about the same time as the maximum flow may be expected through the manhole covers. Just before the joint outlet sewer in Northeastern New Jersey was

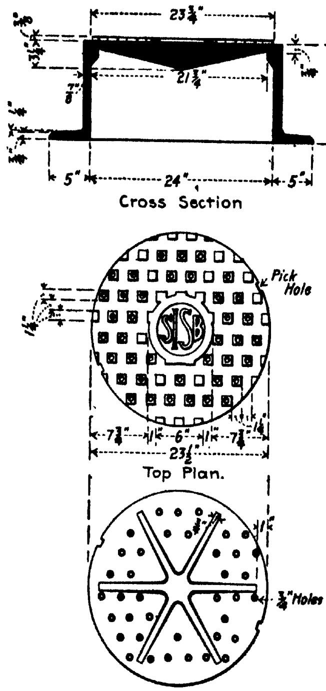

Fig. 234.—Syracuse frame and cover.

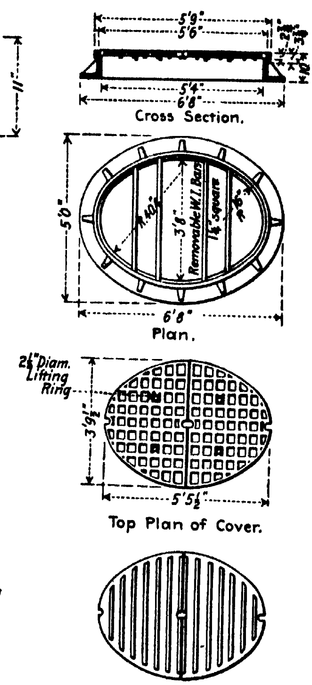

Fig. 235.—Brooklyn frame and cover.

completed, there was a very heavy storm. There were 125 miles of sewer in the system at that time, and about 1965 manholes having ventilated covers. No catch-basins were connected and no roof water was supposed to be admitted. According to the chief engineer, Alexander Potter, as nearly as could be ascertained 3,000,000 gal. of water entered the system in 24 hours through these covers, or an average of 1.1 gal. per minute per manhole.

The standard manhole cover of Syracuse, shown in Fig. 234, resembles that used in New York, except that the perforations of the covers are much more numerous and only half as large, and the surface has an entirely different pattern. This pattern was long a favorite, but has recently been criticized by street department officials as more slippery than that shown in Fig. 232. It is also used in the very large manhole frame and cover shown in Fig. 235, illustrating the

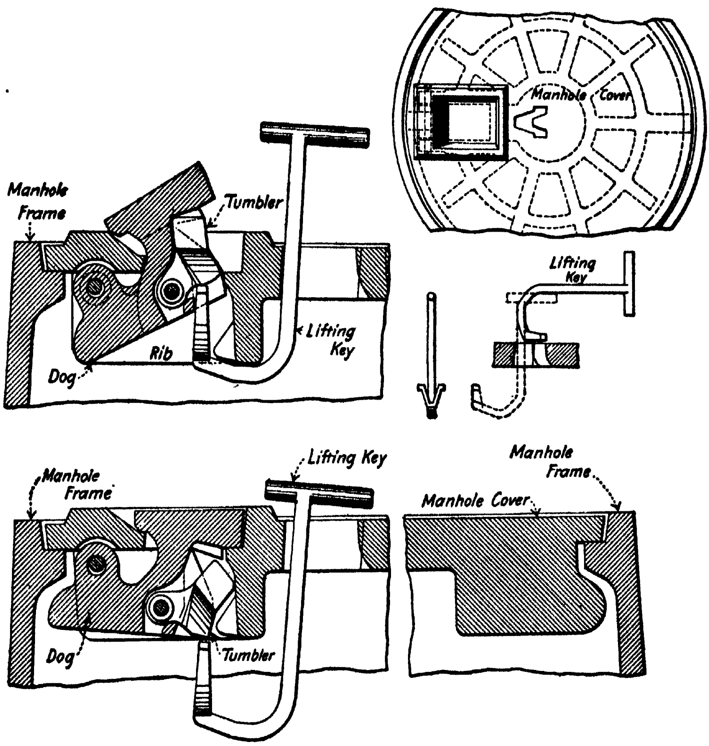

Fig. 236.—Locking device for manhole or catch-basin cover, Boston.

castings used at the top of the wellhole shown in Fig. 224. It will be noted that with such a large cover the casting is made in two parts.

The locking device for manhole and catch-basin covers shown in Fig. 236 was designed and patented by R. J. McNulty, mechanical engineer of the Sewer Service of the Boston Public Works Department. The lock is a cast-iron dog hung loosely on a pin, with a lug projecting

rearward to engage the underside of the ledge of the manhole frame. The dog also has an arm projecting forward to form a closure for the opening in the cover. Beneath this arm are three ribs, between which

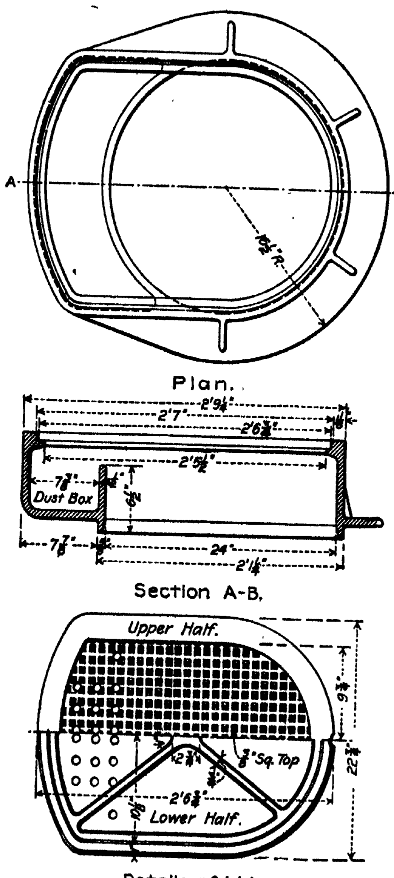

FIG. 237.—Ventilated manhole cover and frame (Kirkpatrick).

are pivoted two tumblers so shaped that gravity causes them to lie normally under the ledge of the cover. It is then impossible to unlock the cover because the tumblers engage the ledge on it and prevent the lug on the dog from disengaging the ledge on the frame. The unlock-

ing can be accomplished by inserting the two-pronged lifting key through the special hole in the cover, turning it 180 deg. and then lifting it, which will lift the tumblers and dog, allowing the cover to be removed. When the cover is replaced, the dog and tumblers fall by gravity into position and lock the cover automatically. The cover cannot be unlocked with a bent wire, like many locking covers. Many of this type are used in Boston to prevent the dumping of ashes and other refuse into manholes and where the displacement of a cover would be particularly dangerous to traffic.

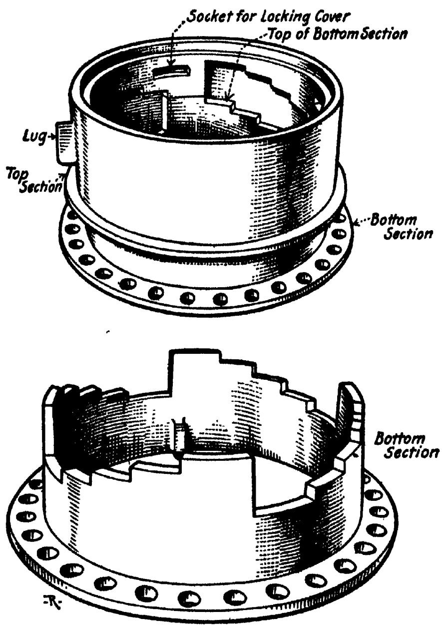

FIG. 238.—Adjustable manhole frame (patented).

A type of ventilated manhole cover and dirt box, used in a number of sewerage systems, designed by Walter G. Kirkpatrick, is shown in Fig. 237. The designer states the advantages of this form as follows, in *Eng. News*, Aug. 26, 1909:

"The dust box is durable, easily cleaned, cannot drop itself nor its contents into the sewer, and allows easy access to the manhole and the sewer for inspection; also the casting is immovable on its setting and presents a neat appearance on the street. It is designed for either paved or unpaved streets and for light street traffic."

Mr. Kirkpatrick stated that the cost of the additional weight due to the dust box was but little more than that of a sheet metal dust pan, and the cast-iron dust box he regarded as much more permanent and convenient. The average weight of the frame and cover was about 425 lb.

For streets where heavy travel wears the pavement rapidly, an adjustable manhole frame which can be readily made lower is desirable. A form used in Boston and its vicinity is shown in Fig. 238; it was designed and patented by E. S. Dorr, Chief Eng. of Sewer Service, Boston Public Works Department. The frame is in two cylindrical sections. The bottom section has four steps in its wall and the top section has a series of inverted steps cast around the inside of the cylinder. The frame can

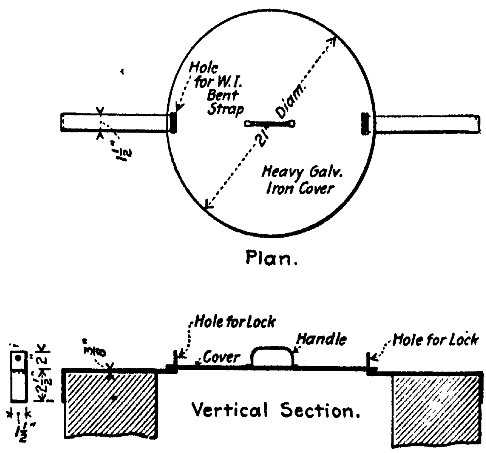

FIG. 239.—Locked manhole cover, Philadelphia.

be lowered 4 in., in four drops of 1 in. each. The bottom section is attached firmly to the masonry of the manhole while the top section is prevented from turning by a lug which is held fast by the pavement. It is stated by C. H. Dodd of the Boston Sewer Service that in some cases concrete manholes are finished with two courses of brick masonry at the top, because this makes it easier to adjust manhole frames to the changes in elevation of pavements which occur from time to time. Chipping off the top of a concrete manhole for this purpose is a tedious process.

It is sometimes necessary to lock the entrance to a manhole more certainly than can be accomplished by any of the catches which were at one time used to some extent to prevent the removal of the covers. A cover for the purpose is shown in Fig. 239. It is used at the top of a

wellhole, in Philadelphia, in which a gaging machine is kept, and immediately on top of it rests a manhole frame and tight cover of the type illustrated in Fig. 233. The cover is a circular plate 21 in. in diameter, and is carried by two flat wrought-iron bars ⅜ in. thick, which are bent at each end. There is a ½-in. hole for a Yale lock in the end of each bar, which is bent up to fit into a hole cut for the purpose in the cover.

A watertight manhole frame and cover, designed for the sewerage system of Concord, Mass., by one of the authors, is shown in Fig. 240.

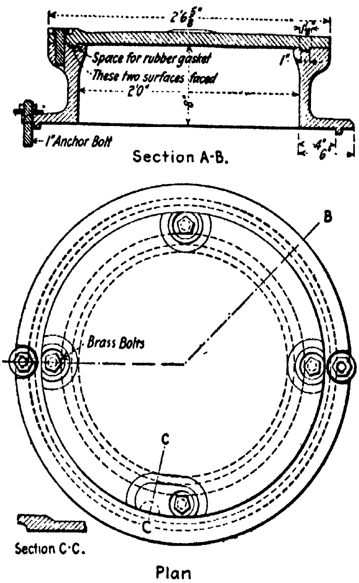

Fig. 240.—Watertight manhole frame and cover.

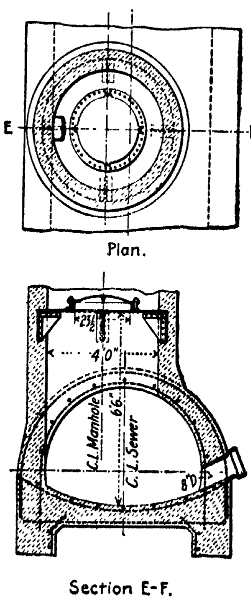

Fig. 241.—Watertight manhole diaphragm.

No detailed explanation of this design is necessary, except that the brass bolts were equally spaced so that the cover will fit in any of four possible positions. Fig. 241 shows how a manhole frame and cover of this type were used inside a manhole in Louisville, where it was necessary to provide against flooding neighboring lands with water from the sewer, when the river into which it discharges is at a high stage.

Lampholes.—It has been intimated already in several places that the authors have not found occasion to use lampholes on their work. In cases where they might have been employed, it was considered that the

additional cost of a manhole was well warranted by the advantage of accessibility to the sewer which it presents. It is true that by means of mirrors attached at proper angles to a rod lowered into a lamphole, with a good light reflected down the lamphole by the mirrors, it is possible to see something of the condition of the sewers in its vicinity. The main use of these shafts, however, is to enable a man to lower a light of some sort down into the sewer, so that an observer stationed at a manhole on either side of the shaft can inspect the interior of the pipe. It is frequently stated that a lamphole can be used for flushing, if a hose connected with a nearby hydrant is carefully lowered down it; this may be true, but the authors have never heard of a case where it was done. In their opinion, the best views in this and other countries regarding these lampholes have been well summarized by Frühling in his "Entwässerung der Städte," in the following words:

"In order to economize in manholes these oftentimes alternate with lampholes, which are cheaper to construct and suffice to enable the flow of the sewage to be observed. This can be done either by looking down the shaft after removing the cover, or a lamp can be lowered down the shaft and can be observed from the nearest manhole, either directly or with the aid of a mirror. In most cases the character of the flow will afford information whether everything is as it should be or a clogging has arisen, and whether the cause of the latter is above or below the lamphole. The obstructions are removed from the nearest manhole, for the lamphole permits only a very slight means of ingress, such as the introduction of a hose. As far as the diameter of the lamphole is concerned, from 6 to 10 in. is enough, according to the depth of the sewer. The shaft consists of vitrified clay, concrete, or iron pipe, and more rarely masonry. The frame and cover at the top are to be placed in the roadway so that the weight coming upon them does not bear on the shaft, which would transfer it to the pipe sewer. If the lamphole is at a place where a flat grade changes into a steeper one, the cover should have ventilating holes."

In addition to what is stated in this quotation, it is desirable to lay emphasis on the necessity of avoiding any weight on the shaft. Experience shows that even the weight of the riser pipe forming the shaft will sometimes break the sewer pipe from which it rises. The disastrous experience of this sort at Memphis, mentioned in the Introduction, has been duplicated at many other places. Consequently the frame and cover, which are made like small manhole castings, should be carried by a ring of concrete or masonry surrounding but not touching the vertical pipe. Even with such precautions a lamphole is bound to be a source of structural weakness, and its use should be avoided if possible.

CHAPTER XV

JUNCTIONS, SIPHONS, BRIDGES AND FLUSHING DEVICES

Junctions.—The earliest discussion of the importance of easy curvature and of carefully guiding together the streams of sewage at a junction, which the authors have found, appears in the report of the British General Board of Health of 1852, where Roe, best known for his table of the areas drained by circular sewers of different diameters, made this statement:

"Every junction, whether of a sewer or drain, should enter by a curve of sufficient radius; all turns in the sewers should form true curves, and as, even in these, there will be more friction than in the straight line, a small addition should at curved points be made to the inclination of the sewer. I may mention a case or two in illustration. In 1844, a great quantity of rain fell in a short space of time, overcharging a first-size sewer and flooding much property. On examination, it was found that the turns in the sewers were nearly at right angles, and also that all the collateral sewers and drains came in at right angles. The facts and suggested remedy were reported to the Holborn and Finsbury Commissioners, and directions given by them to carry out the work. The curves and junctions were formed in curves of 30 ft. radius, and curves with cast-iron mouths were put to the gully-chutes and drains; the result was that although in 1846 a greater quantity of rain fell in the same space of time than in 1844, no flooding occurred, and since then the area draining to this sewer has been very much extended without inconvenience. In another case flooding was found to proceed from a turn at right angles in a main line of sewers. This was remedied by a curve of 60 ft. radius, when it was found that the velocity of current was increased from 122 (as it was in the angle part) to 208 (in the curved part) per minute, with the same depth of water."

With small sewers which it is impracticable for a man to enter, the changes in direction as well as grade must be made in manholes, as already explained, or at lampholes. If this is not done there is a risk of a stoppage occurring at some point where its location cannot be accurately determined, and if such a thing occurs the only remedy is to dig down through the street to the sewer. By the time the obstacle has been removed, the sewer repaired and the trench filled, the desirability of avoiding such occurrences in the future will be entirely clear.

Where the sewers are large enough to be entered, so that their junctions do not need to be made in manholes, and they come together with a

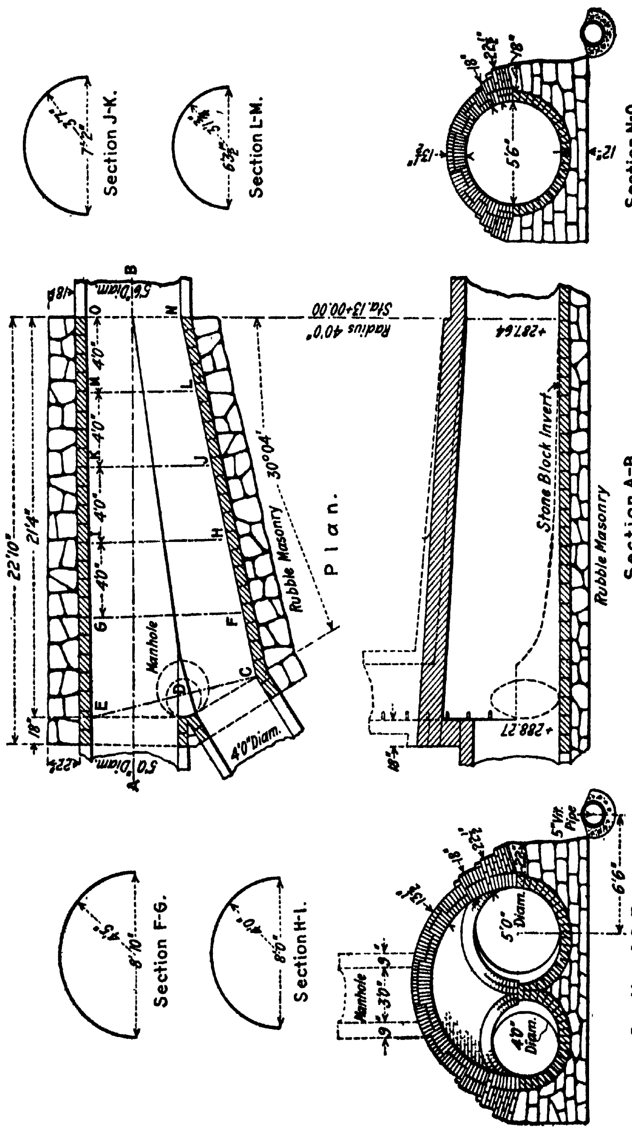

FIG. 242.—Bellmouth junction, Philadelphia.

horizontal angle between their axes less than about 30 deg., a special structure called a junction is required. For many years these junctions were usually of the type shown in Fig. 242, and were called "bellmouths" or "trumpet arches." The two sewers are constructed as independent channels until the outside lines of their masonry come together at the springing line of the arches. If they were continued beyond this point as independent arches, the tongue forming the support for both arches would gradually become thinner and thinner, and the roof of the junction would consequently be in danger of falling through lack of supporting strength at this point. Eventually the tongue would become so thin that even the most reckless builders would not try to carry the roof upon it. Accordingly where, at the springing lines, the outside of the arches come together, no further attempt is made to have the upper portion of the confluent sewers independent, but a large arch is thrown across the two. At the highest point of this arch, just in front of the brick wall which closes the large end of the structure, a manhole or ventilating shaft of some sort is frequently erected. Great care should be paid to forming the curves of the invert to the correct lines, because at these junctions there is frequently some sedimentation, due to backwater, and the inverts should offer no obstruction to the washing away of these deposits by the first storm that arises.

The structure shown in Fig. 242, was built of brickwork, but bellmouths are frequently constructed of concrete. Where brick is employed and the masons are experienced men, the construction of one of these junctions, even when more complicated than that illustrated, is not a difficult task; while the centers must be strong, they do not require the careful finish of a form for concrete, such as that shown in Volume II, which was used on a structure in Louisville. In any case, however, the expense for one of these bellmouths is largely made up of skilled labor, either in laying the brick or in making the forms. To avoid, so far as possible, any further increase in these items, some engineers have recently turned to flat-topped junctions.

A flat-topped junction constructed in Pittsburg is shown in Fig. 243. It is a structure which is more expensive to build than most of the same general type, because it was inserted on an existing brick sewer of large size, which it was desirable to disturb as little as possible. This was rendered more easy from the fact that the sewers come together at an angle of about 45 deg., which renders unnecessary a long tongue at the junction of the invert. The roof, in this case, is a reinforced concrete slab, and the manner in which the old brickwork has been surrounded with concrete, so as to utilize it as fully as possible, deserves attention. The sharp pitch given to the new sewer, where it joins the large existing

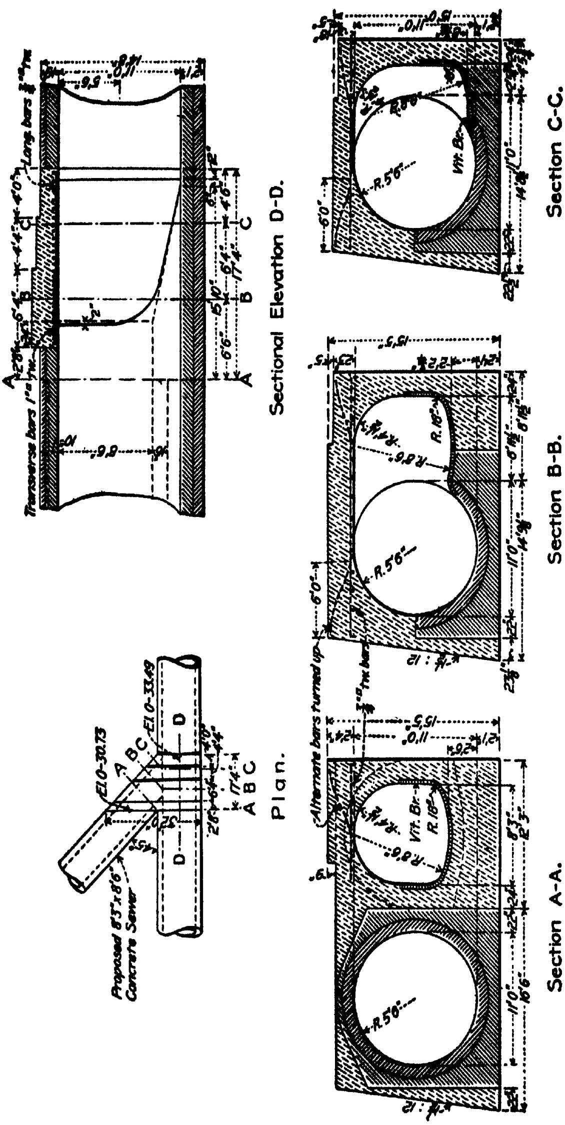

FIG. 243.—Flat-top junction, Pittsburg.

sewer, also deserves attention, and this feature of design will be referred to a little later.

Where flat-topped sewers are necessary at junctions, and the angle which the axis of the two confluent sewers make is small, it is now customary for the roof to be carried by I-beams. The construction of a junction of this sort in Philadelphia, may be mentioned as an illustration of the general arrangement. There were two brick sewers, 10 and 11 ft. in diameter respectively, which came together with inverts at the same elevation. Both were of the brick and rubble type used so extensively in that city. The total length of the interior of the junction structure was 45 ft. The cross-sections of the invert were worked out in the usual manner. Where the side curves of the circular arc of the invert finally became vertical, the walls of the junction were run up straight and given a thickness of 3 ft. 6 in. The minimum depth of concrete below the stone block invert was 12 in. The steel beams resting on top of the side walls, were spaced 3 ft. apart on

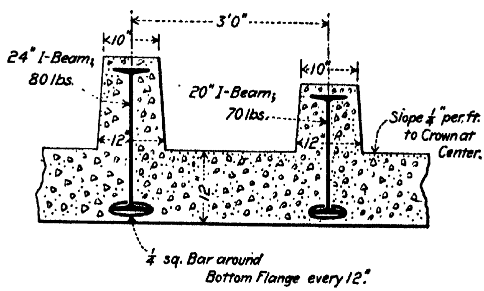

FIG. 244.—Roof detail, Philadelphia junction.

centers. The longest was 25 ft. and was a 24-in. 80-lb. section. The shortest was 18 ½ ft. long and was an 18-in. 55-lb. section. Fig. 244 is a detail of the roof showing the construction. Both the form work and labor for a roof of this sort will probably be less expensive than with masonry bellmouths, but the cost of the steel beams may influence the total cost of the structure so that it will not be as cheap as one of the older types. Sections of this general nature deserve more attention, however, than has been paid to them, for it is not impossible that by careful study a standard structure with a slab top may be developed which will prove decidedly economical.

There are certain theoretical features connected with the design of these junctions which should always be kept in mind, although it is a common experience that it is impossible to satisfy all theoretical requirements in work of this nature, and the best the engineer can do

is to effect a compromise which will result in a structure of ample strength and fitness for the average demands of service. These theoretical considerations have been summed up by Frühling, as follows:

"Sewers must be joined in such a way that no decrease in velocity occurs, because that will result in the subsidence of the silt and suspended matter. It is as necessary to avoid, therefore, a widening of the channel as the formation of an obstruction to the flow, and the two channels should gradually blend into each other, but with the elongations and grades of the inverts so arranged that the discharges from the individual branches have the same surface elevation at the point of junction. With corresponding rising and falling of the sewage in the sewers which are brought together thus, it would be possible to base the designs on any proportion of the capacity of the sections being utilized, but as the surface of the sewage in the trunk sewer is ordinarily proportionally higher than that in the laterals, the engineer is compelled to select arbitrarily some proportion of the full capacity, as that which will be utilized, and remember that an excess use of the capacity will cause the additional height in the trunk sewers to back up the sewage in the branches discharging into it (except those discharging close to its crown). The smaller the available difference in elevations, and hence the flatter the grades, the lower should be the proportion of the full capacity which is chosen as the basis of the design, but it must not be below a proportion which corresponds to the discharge of the average dry-weather sewage, in order that the backwater may be limited to the periods of flow of the maximum dry-weather sewage and of the storm water. With better grades, the design can be based on larger volumes of water, such as the maximum dry-weather discharge or a definite dilution of it by storm water; the upper limit corresponds to the run-off of heavy storms. In this case, assuming that the sewers run full, the crowns of the sewers are to be brought to the same elevation but the inverts will be at different elevations, corresponding to the heights of the different sewers. In all lower stages, the sewage in the branches will enter the trunk sewer through a short section having an increased grade.

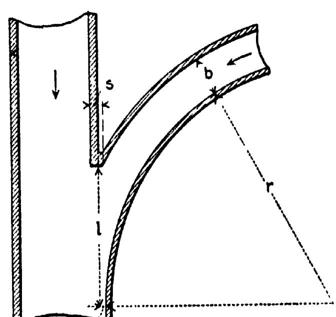

FIG. 245.

"The length, l, Fig. 245, of the junction, depends upon the radius, r, and the width, b, of the branch sewer, the increase in width, m, of the trunk sewer, and the thickness of the masonry, s, at the junction. Then

$$l^2 = (r + b + 0.5s)^2 - (r + m)^2$$

This shows that a change at the junction to a section of greater width, as from an egg-shape to a semi-elliptical shape, reduces the length of the

junction. So far as the value of r is concerned, it is never taken at less than $5b$ in the better class of designs; the resistance to the flow of the sewage increases as the radius decreases, but the resulting loss in fall is only slight."

In large cities the junctions are not always such simple affairs as those shown in Figs. 242 and 243. In Fig. 246 a complicated junction in Philadelphia is illustrated. Here there is a brick sewer 9 ft. in diameter crossing a brick sewer 8 ft. 3 in. in diameter, and the problem is to put in junction chambers and sanitary sewers in such a way that the course of the larger sewer, beyond this intersection, will serve as a relief for the storm water from the smaller sewer, and that the dry-weather sewage in the latter will flow into the channel which will also carry away the dry-weather sewage from the former. This was accomplished by four junction chambers and two 30-in. cast-iron pipe sewers, shown in the illustration. It will be observed that a very large proportion of the section of the 9-ft. sewer will be utilized before there is any discharge from it into the overflow sewer, while in the case of the 8¼-ft. sewer everything that is not dry-weather sewage will be immediately discharged into the overflow outlet.

SIPHONS

Unfortunately there are not two words in the English language to make a sharp distinction between what we call inverted siphons, "Düker" in German, and true siphons, "Heber," in German. Consequently engineers frequently speak of siphons when they mean inverted siphons, and considerable confusion sometimes arises on this account. The difference between the two classes of structures is as great as that between the North and the South, however.

Where a conduit has a V-form in its profile between two points, that is to say, is provided with a descending and then a rising leg, it forms an inverted siphon. This may or may not have such a bend that the liquid in the bottom will always seal the legs like a trap. Where there is no such seal, the inverted siphon is commonly spoken of as "incomplete;" a complete inverted siphon is really a large trap, duplicating on a great scale the apparatus so familiar on a small scale in plumbing.

A true siphon, on the other hand, consists of a rising leg followed by a falling leg, the two having an A-form and serving to raise water above the hydraulic gradient between two points on a conduit, by utilizing atmospheric pressure. The siphon must discharge at a lower elevation than that at which the liquid enters into it, and the maximum theoretical height over which the siphon is able to lift water is $(32 - H)$ ft., where H is the head in feet necessary to give the liquid its velocity. If air or gas collects at the summit of the siphon, it will eventually interrupt the service and on this account various devices are used to guard against this danger. It is particularly important in the case of siphons operating

with sewage, because of the tendency of gases to be given off by the sewage as the pressure to which it is subjected toward the summit of the siphon becomes reduced.

Inverted Siphons.—Where an inverted siphon flows full of sewage, small increases of discharge in the sewer proper become suddenly checked upon reaching the inverted siphon. There is a tendency for the suspended matter and silt to be deposited. It is therefore desirable to provide as well as may be for maintaining relatively high velocities by confining the flow to one or more restricted channels and allowing additional slope through the siphon. In some cases catch-basins or grit chambers have been built just above siphons, but these are troublesome to clean and the material removed from them is usually very offensive. Siphons should be flushed frequently and their operation inspected regularly. Where the inverted siphon forms part of a pressure main, these considerations lose their importance, of course, for then the inverted siphon merely carries a little greater pressure at its lowest point than other parts of the line.

Since an inverted siphon is subjected at all points of its cross-section to an inner pressure, the walls must be in tension, although the amount of tension may be modified if the exterior of the inverted siphon is subjected to water pressure or the pressure of earth. On account of these tensile stresses, inverted siphons are usually constructed of steel, iron, reinforced concrete or wood-stave pipe heavily banded. The use of these materials in sewers is discussed in Chapter X.

The computation of the sizes of pipe for inverted siphons is made in the same way as that of sewers and water mains; their diameter depends upon the grade and the maximum quantity of water to be carried. The latter depends, in the case of inverted siphons under rivers, upon the presence or absence of a storm overflow before the inverted siphon is reached, and on the degree of dilution of the sewage before the overflow outlet comes into service.

As an example, a case in Frühling's "Entwässerung der Städte" may be repeated here. The problem was to deliver 31,783 cu. ft. of sewage an hour, by means of an inverted siphon 492.1 ft. long, with an available fall of 1.968 ft. when the sewage is diluted with twice its volume of storm water; at this stage, the relief outlet above the siphon begins to discharge. The sewer running to the upper end of the inverted siphon is a 72/48-in. egg-shaped section with an invert grade of 1 :300; this grade is also that of the sewer connected with the outlet of the inverted siphon.

An hourly flow of 31,783 cu. ft. of sewage, with twice as much storm water added, corresponds to 26.486 cu. ft. per second, which is to be carried off on a fall of 1.968 : 492.1 = 1 : 250. A fall of 1 : 100 would correspond (by using the method explained in Chapter I) to a discharge of

$$26.486 \sqrt{(250/100)} = 41.887 \text{ cu. ft. per second,}$$

which, Table 3 shows, will require a pipe 33 in. in diameter, or two pipes 22 in. in diameter. It is true that a 22-in. pipe on a 1 : 100 grade has a nominal capacity of but 17.94 sec.-ft., but it will require only a trifling excess head to make it carry the required quantity at the wet cross-section of maximum discharge, which is not that of the entire pipe, as is shown in Fig. 133. The necessary grade for the maximum discharge will be

$$H = \frac{20.944^2}{19.26^2} \frac{1}{250} = \frac{1}{211}$$

and the excess head will be 492.1/211 − 1.968 = 0.36 ft.

In discharging 26.486 cu. ft. per second on a grade of 1:300 the 72/48-in. egg-shaped sewer will be filled to a depth of about 2.067 ft., at which elevation the sill of the relief outlet should be placed; it is also the datum for fixing the elevation of the invert of the sewer at the other end of the inverted siphon. It should be 54/36 in., since at the moment the relief outlet begins to discharge this sewer must also be carrying 26.486 cu. ft. per second. This will bring the water surface to an elevation of about 1.68 ft. above the invert. As there is a drop of 1.968 ft. according to the original assumption, and one of (1.968 + 0.36) = 2.328 ft., if two 22-in. pipes are employed, then the invert of the sewer running from the inverted siphon must be 1.68 + 1.968 = 3.648 ft. below the sill of the storm overflow, if a 33-in. pipe is used, or 1.68 + 2.328 = 4.008 ft. if two 22-in. pipes are used.

Fig. 247, which shows a structure on the sewerage system of Louisville, is introduced to illustrate the manner in which a bypass can be constructed to discharge the sewage into a neighboring creek or other body of water when the inverted siphon requires cleaning. This structure includes two 12-in. iron pipes carried under the creek on the same invert grade as that of the sewer. In addition to these there is provided a 36-in. iron pipe dipping down from the grade of the sewer beneath the bottom of the creek. This pipe will act as an inverted siphon, but will not be put into use until the flow in the sewer exceeds the combined capacity of the two 12-in. pipes. At each end of the crossing there is a concrete chamber giving access to the siphon to facilitate cleaning when it is found to be necessary. There is also an emergency outlet to the creek, through which the sewage may be turned when the siphon is being cleaned or repaired. A sluice gate in the outlet chamber also makes it possible to shut off any backwater from the intercepter at such times. The concrete protection of the pipes has its top on the level of the bottom of the creek.

A longer structure, also on the Louisville sewerage system, is shown in Fig. 248. This is on the line of a 48-in. sewer and consists of vitrified pipe encased in concrete. At the inlet chamber, the arrangements are such that any one or two or all of the pipes may be put in service, according to the quantity of sewage flowing. It was the intention of the designer to confine the entire flow to the 18-in. pipe so long as the quantity of sewage did not exceed its capacity, and then to substitute

with sewage, because of the tendency of gases to be given off by the sewage as the pressure to which it is subjected toward the summit of the siphon becomes reduced.

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An hourly flow of 31,783 cu. ft. of sewage, with twice as much storm water added, corresponds to 26.486 cu. ft. per second, which is to be carried off on a fall of 1.968 : 492.1 = 1 :250. A fall of 1 : 100 would correspond (by using the method explained in Chapter I) to a discharge of

$$26.486\sqrt{(250/100)} = 41.887 \text{ cu. ft. per second,}$$

one of the 30-in. pipes. Other changes can be made from time to time so as to provide the necessary increase in capacity to meet the growth of the city. The entrance to each pipe is controlled by a sluice gate set in the masonry and also by stop planks and overflow chambers, so that in case of emergency the sewage will flow automatically into a second or third pipe when the one in use is overcharged. Provision is also made

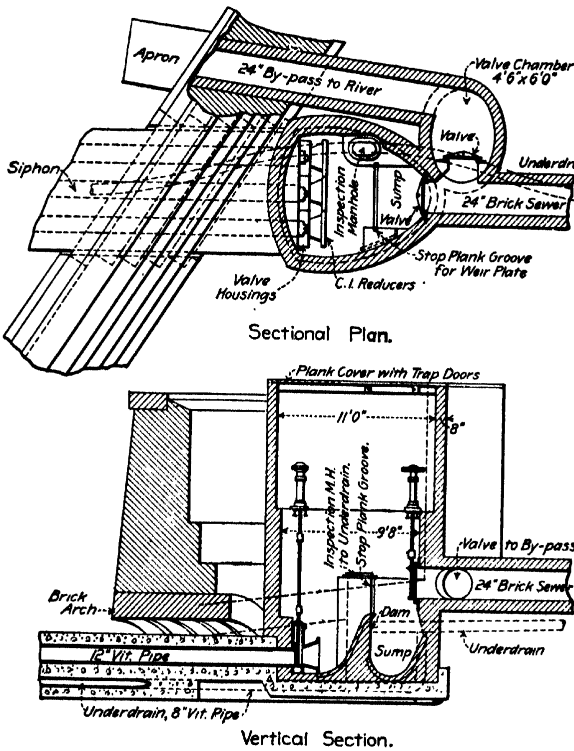

FIG. 249.—Inlet chamber, Woonsocket inverted siphon.

for an automatic overflow into the neighboring creek, and if it is necessary the entire discharge of the sewer may be turned for a short time through a 30-in. blow-off conduit into the creek. At the outlet chamber, any or all of the pipes can be closed by means of stop planks.

At the lowest point of this siphon a third chamber is provided for the purpose of draining and cleaning any of the pipes. For this purpose, the sewage is drawn off into a sump and then pumped into the creek,

after which a section of the pipe 4 ft. long can be removed and the line running from it to either chamber can be cleaned in the usual way.

There are a number of inverted siphons in the Woonsocket sewerage system, which was designed by Frank H. Mills, city engineer, with the advice of Dr. Rudolph Hering, consulting engineer. A typical structure at this place consists of three lines of 12-in. pipe placed 3 ft. c. to c. and embedded in concrete. These pipes were laid across the river by means of a coffer-dam. It will be noticed in Fig. 249, that the invert of the 24-in. sewer running into the inlet manhole is about 4 ft. above the invert of the end of the inverted siphon and in the bottom of the chamber is a sump into which the sewage drops. This sump is separated from the rest of the manhole by a low dam and weir over which the sewage flows to the pipes forming the inverted siphon. The weir has a crest of thin copper plates. Immediately adjoining this chamber is a valve chamber connecting the 24-in. brick sewer with a 24-in. by-pass to the river. The sewage is diverted through this by-pass when it is desired to clean out the inverted siphon. There is a retaining wall of granite rubble laid in cement, at the inlet and the outlet chamber, and in the wall there is a brick arch over the pipe in order that no weight may come upon the latter and crush them. Another detail to which attention should be called is the manner in which the underdrain has been swung to one side as it passed under the inlet chamber, and has been provided with a small inspection shaft.

There are several river crossings on the sewerage system of Concord, Mass., each consisting of a line of 12-in. cast-iron pipe. At the head of two of these there are flushing chambers for accumulating sewage and discharging it intermittently in large quantities in order to keep the pipe clean. Each chamber is built of brickwork and has a dome roof Fig. 250; one is 20-1/2 ft. in diameter and discharges from 15 to 20 times in 24 hours, and the other is 10-1/2 ft. in diameter and discharges 8 to 10 times in 24 hours. The chambers are discharged by means of Van Vranken automatic siphons.

There are a number of inverted siphons crossing Paxton Creek in Harrisburg, Pa., in order to deliver sewage to an intercepter built in 1903 from the plans of James H. Fuertes. The connections at both ends of these siphons are shown in Fig. 251, from *Eng. Record*, Oct. 11, 1902. At the inlet end of each, a section of the existing sewer was taken out of sufficient length to permit the construction of a new manhole, sump and connection with a silt basin. The dry-weather sewage as it comes down the old sewer runs down a cast-iron pipe leading from the sump in the sewer invert to the silt basin, which has a depth depending upon the conditions encountered at each crossing. The two outlets from this basin are 3-1/2 and 4-1/2 ft. above its bottom, and the sewage flows through them and down under the creek in two lines of cast-iron pipe, rising at

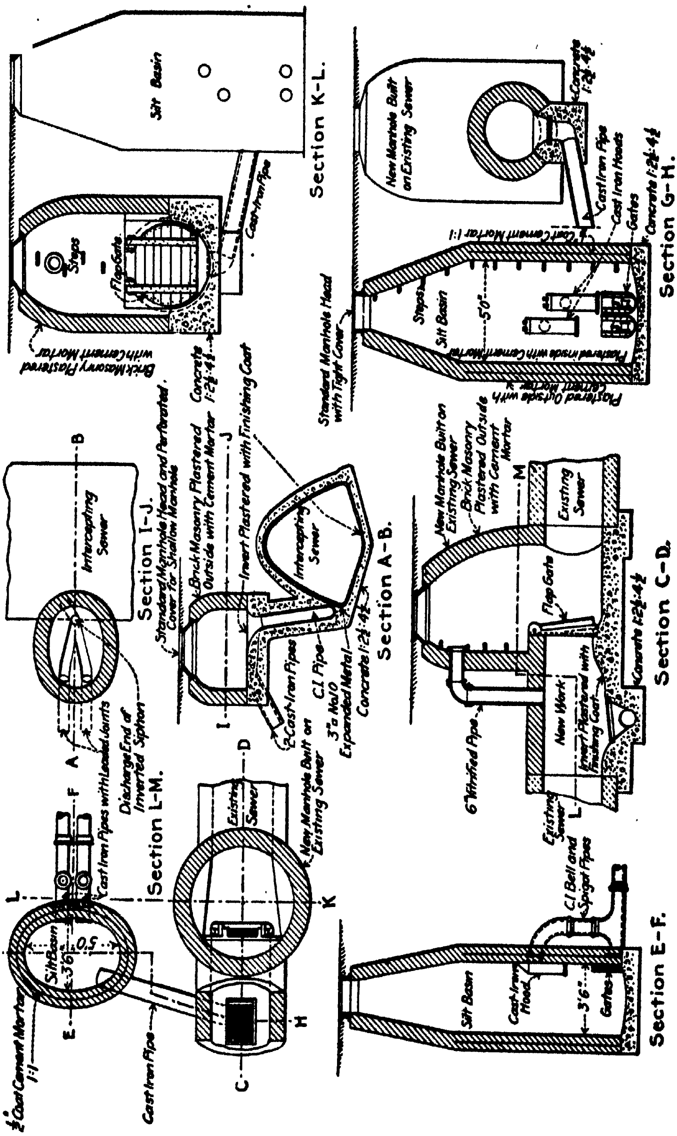

FIG. 251.—Inverted siphon at Harrisburg.

the other side in a shallow manhole, from which it is discharged into the intercepter through a cast-iron drop pipe. The sewers are built on the combined system, and the entrances to the inverted siphons were designed to permit the greater part of the storm water to pass directly into the creek through the old outlets, flap gates being provided just beyond the sump to prevent backwater from entering the intercepter in times of flood. These gates were made of cypress lumber in order to secure lightness, and were faced with rings of steel where they bore upon the cast-iron frame. Each gate was hung on two wrought-iron straps, extending its entire width and sunk into the lower side of it. After the gate had been hung and closed the face joint was made by pouring lead into a groove left in the face of the frame for that purpose.

The sump at the intake end of the inverted siphon, through which the sewage enters the silt basin, is protected by a cast-iron grating. The outlets from the silt basins are provided with cast-iron hoods to prevent floating matters from getting into the inverted siphon, and sluice gates at the bottom of the basin afford means for cleaning the pipes. The two pipes under the creek unite at the discharge end in a manhole, from which the sewage flows down a cast-iron pipe into the intercepter.

An inverted siphon shaped like a Venturi meter to prevent deposition of suspended matter by an increase of velocity without appreciable loss of head, has been built on the 39th St. conduit under the Illinois and Michigan Canal in Chicago. The top of the conduit required lowering 10 ft. to permit the necessary 4 ft. 8 in. of water in the canal. The section of the conduit was gradually changed from an ellipse on end, 14 ft. high and 12 ft. wide, to one on its side, 9 ft. high, and then back again. The throat section of the siphon is about 65 per cent. of the full section, and it was estimated that the loss of head would be less than 0.03 ft., while the velocity would be increased to about 3 ft. per second, which was considered a transporting velocity for the material likely to reach that point. The inverted siphon is 200 ft. long, 12 in. thick and constructed of 1:2-1/2:5 concrete reinforced with 1/2-in. steel rods forming hoops 6 in. apart and longitudinal ties 12 in. apart.

A typical short inverted siphon is shown in Fig. 252. It carries the contents of a large drain under the outfall sewer at Baltimore, and has two 6-ft. circular conduits and one 14-in. pipe, all ending on each side in an enlarged chamber. The upstream end contains two grit wells, separated by a wall rising 2-1/2 ft. above the bottom of the invert of the sewer. Either grit well can be shut from the sewer at the height of this division wall by means of stop planks. The ends of the grit wells toward the inverted siphons are closed by curved dams, the tops of which are 1 ft. below the top of the division wall. The 14-in. cast-iron pipe is in the center line of the sewer and can be shut off from connection with either grit well by stop planks between the division wall and the end

of the curved dam. The intake of the 14-in. pipe is 1-1/2 ft. below the crest of the dam and 2-1/2 ft. below the top of the division wall. In

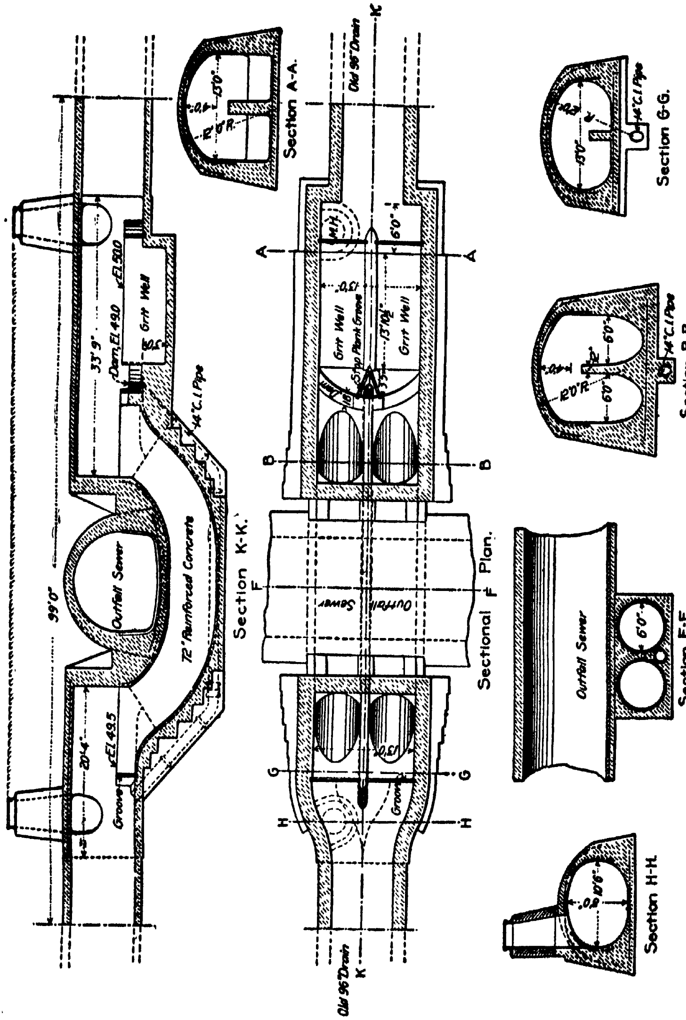

FIG. 252.—Inverted siphon on storm-water drain, Baltimore.

ordinary operation the pipe carries the flows, and when these exceed its capacity the sewer discharges into one of the 6-ft. siphon

pipes over the dam at the end of one of the grit wells. Should the storm become more severe, the 14-in. pipe and both 6-ft. conduits will be put in operation. The normal arrangement is to place stop planks across one end of one of the grit wells and between this grit well and the opening of the 14-in. pipe. The sewage then flows unobstructedly to the other grit well and into the 14-in. pipe. When it rises high enough it overtops the dam at the end of the grit well left open, and when it rises a foot higher overflows the stop planks at the head of the other grit well and division wall and discharges through both of the inverted siphons. On account of the custom of sweeping street refuse into the storm-water drains, the structure is required to work under very trying conditions, but it has operated successfully since its completion.

One of the largest inverted siphons for sewage is used in carrying a low-level intercepting sewer under Wissahickon Creek in Philadelphia. This is illustrated in Fig. 253. The inlet chamber receives the discharge from a brick sewer 3 ft. 6 in. in diameter, laid on a grade of 0.3 ft. in 100 ft. The inverted siphon is 152.5 ft. long from the inlets of the cast-iron pipe to their outlets. There are a 12-, 20- and two 16-in. pipes, provided at the inlet end with gates so that only those will be in service which it is considered desirable. By using flap gates of the same design, except for boring holes in some of them in bosses to form connections for fire hose, in chamber C and chamber D, a considerable standardization of details has been accomplished.

Siphons.—One of the oldest true siphons on a sewerage system crosses the St. Martin canal in Paris. At this place there is a masonry arch bridge, the Pont Morland, and the siphon is attached to one face of it, forming a semi-circle with a diameter of 52-1/2 ft. Its crown is a little more than 26-1/4 ft. above the sewer leading to it. The gases which are given off from the sewage rise to the top and are led away through a riser 49.2 ft. high, from which they are drawn by an ejector worked by water admitted and shut off at the right times by a float. The siphon can be put in operation by means of the ejector in about 5 minutes, so that any serious interruption in its service is regarded as unlikely. French engineers have made tests of this siphon, which have shown the surprising fact that with a velocity of flow of 3.9 to 4.9 ft. per second, the collection of gases at the crown no longer takes place.

It is generally believed that the first sewage siphon in the United States was constructed at Norfolk, Va., about 1885. It is a cast-iron line 14 in. in diameter, and about 1900 ft. long, which was built by City Engineer W. T. Brooke to avoid very troublesome and expensive trench work in quicksand. The outlet end is provided with a return bend, which prevents the siphon from becoming unsealed, and at the summit there is attached a 2-1/2-in. pipe through which accumulations of

gases and air are removed by means of an air pump at the sewage pumping station. This siphon was in satisfactory operation in 1914, Mr. Brooke informed the authors.

The best-known siphon is probably that constructed at Breslau in 1885, to carry the sewage of a population of about 5000 people from an island in the Oder to the right bank of that river. It is hung from the superstructure of a bridge and is 493.6 ft. long and 5.9 in. in diameter. The highest point of the siphon is at the end of the bridge, and from it the descending leg drops down into a water seal in the bottom of a manhole. At the summit there is a chamber in which the gases are collected. As these gather, the level of the sewage in the chamber gradually falls and finally it reaches such a point that a float inside the chamber operates a water-driven ejector, which sucks off the gases and is finally closed by the rising of the float. This siphon, which was the first of several of the same type in Breslau, although expensive, proved an economical substitute for an inverted siphon which would have been very expensive on account of local conditions.

Extensive use is made of siphons in Potsdam, where one of them has been employed, in fact, as an intercepting sewer. At each point of interception the dry-weather sewage is discharged into a chamber, where it first deposits any silt or sediment in a sump, and then passes over a wall and through a screen into the bottom of the rising leg of a siphon. At the mouth of this siphon there is a sliding valve operated by a float, and somewhat higher in the rising leg there is a ball valve. The float-valve closes the siphon whenever there is a chance that the sump will be drained completely of sewage, and the ball valve is an assurance against the entrance of air. The gases and air are forced out of the siphon by water injected under pressure into the summit. In order to accomplish this the two legs of the siphon must be closed, which is done by means of the valves already mentioned at the inlet end, while at the outlet end, which is at a pumping station, a valve is shut by the attendant before he admits the water under pressure into the siphon pipe. The details of the air-removing chamber at the summit have been worked out so that as the gases are put under a fairly heavy pressure, they lift a heavy valve and escape through small openings into the air. As they escape a float rises on the liquid which replaces the air. This float carries a vertical rod with a needle point at its upper end. When the float has risen to the maximum position, this needle point enters the orifice through which the gases escape, and closes it. This plugs up the passage so that the heavy valve at the top of the passage falls back on its seat. The attendant at the pumping station observes, by means of a pressure gage, when this takes place, and shuts down the machinery which puts the siphon under pressure. There are three points where intercepting sewers discharge into one of these siphons on the Potsdam sewerage

pipes over the dam at the end of one of the grit wells. Should the storm become more severe, the 14-in. pipe and both 6-ft. conduits will be put in operation. The normal arrangement is to place stop planks across one end of one of the grit wells and between this grit well and the opening of the 14-in. pipe. The sewage then flows unobstructedly to the other grit well and into the 14-in. pipe. When it rises high enough it overtops the dam at the end of the grit well left open, and when it rises a foot higher overflows the stop planks at the head of the other grit well and division wall and discharges through both of the inverted siphons. On account of the custom of sweeping street refuse into the storm-water drains, the structure is required to work under very trying conditions, but it has operated successfully since its completion.

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system. A description is given in Frühling's "Entwässerung der Städte."

BRIDGES

The use of bridges in connection with sewers has been fairly infrequent, particularly in the United States. The difficulty has been the strong objection to the use of true siphons for such crossings, and it is rarely possible to support a sewer from a bridge structure unless it is carried up from its position in the street to about the level of the roadway of the bridge, which forms a siphon. It is possible that with more experience with siphons the prejudice against them will disappear.

A river crossing on the joint outlet sewer in northeastern New Jersey, built from the plans of Alexander Potter, is shown in Fig. 254. This

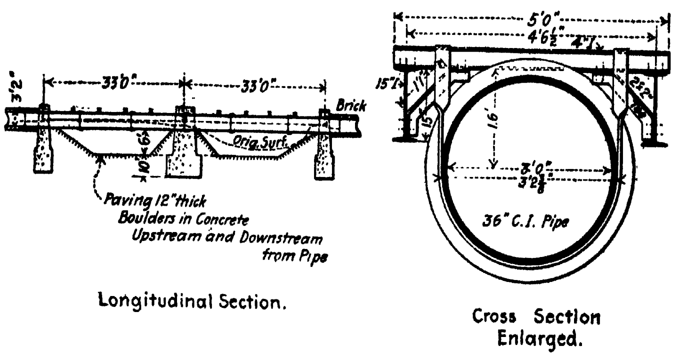


FIG. 255.—Sewer bridge, joint outlet sewer, New Jersey.

is such an elementary structure that it is hardly possible to speak of it as a bridge. The 42-in. cast-iron pipe is supported on posts made of pairs of rails embedded in concrete piers 4 ft. deep and 7 by 4 ft. in plan. There is one of these supports for each length of pipe. This construction was employed in order to minimize the obstruction to the stream flow and secure the greatest possible clearance between supports. It will be observed that the river channel at this place, was widened out considerably so as to afford a greater waterway. A more elaborate structure on the same sewerage system is shown in Fig. 255.

A reinforced concrete sewer bridge was constructed at Morristown, N. J., to carry a 2-ft. sewer across a stream at a point at which there was not sufficient head available to permit the use of an inverted siphon. The stream is flashy and consequently the channel was widened at the site of the bridge and the crossing was made in three spans of 33 ft. each, giving a clear width of 99 ft. without any obstruction other than

two narrow piers. There was some possibility that the structure might be widened and used before long as a highway bridge, and accordingly the girders were made heavier than would otherwise have been the case. The cross-section of the bridge has a width of 4 ft. and a depth of 32 in. The 2-ft. sewer is in the center. This permits the design to be regarded as a pair of girders 12 in. wide and 32 in. deep. This bridge is said to have cost about 20 per cent. less than the bids for a structure consisting of an iron pipe suspended between steel girders.

A 4-1/2-ft. sewer is carried across a canal in Denver, Colo., by means of a reinforced concrete bridge, 44 ft. long, with a clear span of 40 ft. In cross-section it is 4 ft. 8 in. high and 7 ft. 6 in. wide. The circular 4-1/2-ft. sewer is located so that there is 6 in. of concrete below the vitrified brick invert. This gives a cover of about 5 in. above the crown of the section, the top of the bridge having a transverse slope, each way from the center, of about 1 in. The structure is reinforced on each side of the sewer as if both sides were beams, and the total dead load of the span is 93 tons. The design was made by H. F. Meryweather, who considers that a needlessly heavy and strong structure was built, according to a statement in *Engineering Record*, Sept. 7, 1912.

A reinforced concrete structure of a somewhat lighter character was built in Los Angeles in 1907, to carry a 36-in. cast-iron pipe sewer across the Los Angeles River. On each side of the pipe is an 18-in. 55-lb. steel I-beam wrapped thoroughly with 3/16-in. wire surrounded with concrete. Every 36 ft., these beams rest on a reinforced concrete pier which is supported on two reinforced concrete piles. Every 12 ft. the pair of beams are connected by a reinforced concrete diaphragm which forms a support for the pipe.

A box girder sewer approximately 22 in. wide and 34 in. high was constructed in 1910 in St. Louis, across a ravine which it was expected to fill within a few years, but it would take so much time for the fill to settle thoroughly that it was deemed inadvisable to delay the construction of the sewer on that account. The design adopted for this project was a hollow concrete girder (*Eng. News*, Sept. 5, 1912), of two 35-ft. spans with a central pier. The girder was designed to carry the weight of the concrete, the sewage and a triangle of earth on top of the sewer, 3 ft. high. This last provision was to allow for the load which might come on the sewer when the ravine was being filled and before the fill had compacted enough to carry the weight of the sewer.

FLUSHING DEVICES

The primary purpose of flushing is to permit sewers to be laid on flat grades which, while producing adequate velocity to give the desired capacities at the depths assumed in the computations, are not enough

to give at other depths velocities which will carry off at all times all solid matter. The problem of flushing, strictly speaking, is usually merely one of keeping lateral sewers clean from their dead ends to the points where the flow of sewage is great enough to accomplish this without assistance from the water mains. Occasionally the problem is one of furnishing a large volume of water to clean out a main sewer or an inverted siphon. In any case, the object is to increase temporarily the hydraulic gradient in the sewer by means of an exceptional head of water at its upper end. In some European cities the volume of water stored for flushing is quite large, so as to maintain the discharge under this extra

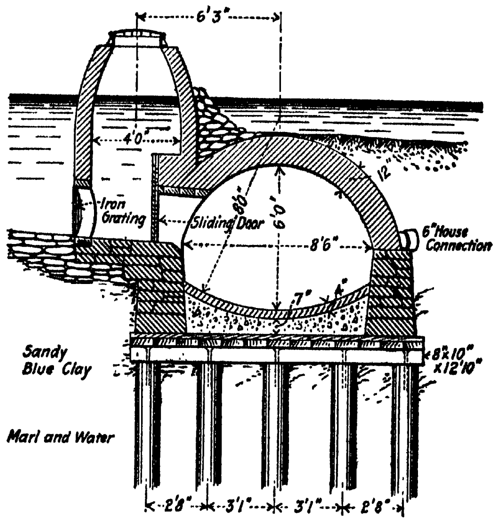


FIG. 257.—Flushing manhole, Minneapolis.

head for a considerable period; in the United States the quantity stored in a flush tank at the end of a lateral sewer is not usually over 350 gal.

Flushing from Brooks.—An example of flushing a large sewer, from a neighboring water course is afforded by the intake on the Harbor Brook Intercepter in Syracuse, designed by Glenn D. Holmes and shown in Fig. 256. The sides and bottom of a brook near this sewer were paved with concrete, and provision was made for temporarily damming the channel with stop planks. The water thus impounded can be diverted through two 15-in. vitrified intake pipes surrounded by concrete, into an 18-in. manhole. This is built of concrete with a vitrified pipe as the shaft, its bell being closed, when not in use, with a stop plank of two thicknesses of 1-in. pine, which can be lifted out of the bell by a chain when

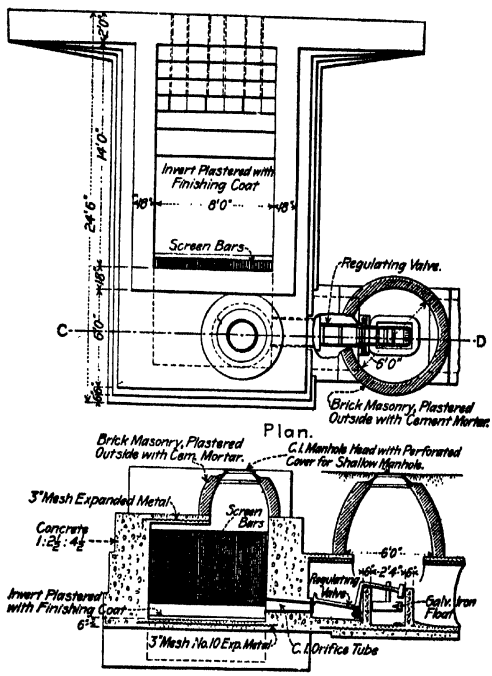
flushing is to begin. From the bottom of this manhole a 24-in. vitrified pipe runs directly into the 33-in. circular concrete intercepting sewer. A manhole is located a few feet farther up the line of the sewer. The difference in elevation between the top of the temporary stop planks in the creek and the invert of the intercepter is about 8 ft.

A flushing manhole built on the Minneapolis sewerage system, 1895, from the designs of Carl Ilstrup, is shown in Fig. 257, from *Eng. Record*, March 28, 1896. This manhole was constructed where a large brick sewer crosses a swamp and in so doing runs through a creek. The ground was very soft and troublesome, and piles were driven on which a grillage was laid below the lowest water level, affording an opportunity to build the sewer inside a coffer. Stone walls were first laid and afterward a mass of concrete was placed between them, of sufficient volume to give the necessary weight for such a structure. At one side of the sewer, the excavation was extended sufficiently to deepen the bed of the creek into a shallow well, which was roughly walled and paved so as to bring its bottom about on a level with the springing line of the brick arch of the sewer. The manhole built up on this foundation had a 2-ft. opening into the sewer, which could be closed tightly by a sliding door. On the opposite side of the manhole was an opening into the creek guarded by iron bars to keep out rubbish. In this way the manhole was kept full of water up to the level of the surface of the creek, and whenever it was desired to flush the sewer the sliding gate between the manhole and the sewer could be opened, admitting creek water in this way under a small head.

A flushing chamber was built at the end of an intercepter constructed in Harrisburg, Pa., in 1903, from the plans of James H. Fuertes, and worked satisfactorily for a considerable time, but was finally practically dispensed with, owing to the admission of brook water at a manhole a short distance below the headworks. It was necessary to use very flat grades in order to avoid prohibitive excavation and pumping, and this grade difficulty was overcome by making the sewer somewhat larger than necessary for the interception of the dry weather sewage alone, and by forming a connection between the upper end of the sewer and the neighboring creek, where automatic regulating gates admitted during dry weather enough creek water into the conduit to keep the flow in all its parts at a self-flushing velocity. During storms, when the lateral sewers were discharging large quantities of both sewage and street water into the intercepter, a float rose which closed the valve and shut out the creek water.

The design of the chamber is shown in Fig. 258. Two sets of three 12-in. vitrified pipes extend through the concrete head-wall as inlets for the creek water, one set 4 ft. higher than the other. The water passes through a large silt basin in order to become free from heavy suspended

matter, and then passes through rectangular cast iron orifices into the regulating chamber proper. The valve regulating the admission of the creek water is of the usual type, the opening being automatically controlled by a galvanized iron float. The rotating arm is attached to the concrete wall of the float well by a short length of angle iron, the hole through which it is bolted to the latter being slotted so as to permit a vertical adjustment. The horizontal leg of this angle and the flanges



Section C-D.

FIG. 258.—Flushing chamber, Harrisburg.

of the trunnion carrying the rotating arm are slotted to allow a horizontal play in two directions. When the valve was installed it was loosely bolted in place, adjusted by means of the slotted holes until it worked perfectly, and then bolted to its final position.

The float well was connected to the sewer by a 4-in. vitrified pipe extending below the invert about 10 ft. down the sewer, where the opening was covered by a cast-iron grating cemented into the bell of the pipe. All parts of the valve with its rotating arm and lever were of

cast iron except the face of the valve and all wearing parts, which were bronze. The galvanized iron float was 11 × 24 in. and 9 in. deep. With the exception of the brick manhole the entire construction was of 1 : 2 ½ : 4½ concrete reinforced by 3-in. No. 10 expanded metal.

In Europe sewers are occasionally flushed by means of the sewage itself. To accomplish this, flushing chambers which contain large gates are employed. These gates are usually open, but are closed when flushing is to be undertaken. After they are closed the sewage backs up behind them and when a sufficient quantity has been stored it is suddenly released by opening the gates, which is accomplished in a variety of ways. Apparatus of this nature has rarely been proposed in the United States. Other methods of keeping the sewers clean are generally preferred and are described in Volume II.

Flushing Manholes.—The flushing of small sewers is carried on either by hand or with the help of automatic apparatus. Opinion seems to be divided regarding the merits of the two methods; the authors' views are stated in Volume II, under the operation of sewerage systems. As a general proposition all flush tanks require some maintenance, and their cost is therefore dependent, in a measure, upon the time spent in inspecting and repairing them. The cost of this time, plus the interest and depreciation on the investment in the apparatus, plus the cost of the water used for the flushing, must be offset against the cost of labor and water where hand-flushing is employed, for the difference in the cost of the manholes used in the two cases is negligible. The amount of water to be used for flushing and the frequency of the flushing depend not only upon the grade of the sewer to be kept clean, but also upon the possibility of dirt finding its way into the sewer.

Hand-flushing is generally done by means of a hose from the nearest fire hydrant, inserted into the manhole at the end of the lateral or on the summit of the sewer to be cleaned. Flushing manholes are also used to a considerable extent. In this case a 1- or 1-1/2-in. branch from the nearest water main is run into the manhole and the entrance to the sewer can be closed with a flap or tripping valve. Water is admitted to the manhole through the service pipe, and when it is full the valve is tripped, allowing the water to rush into the sewer. The same end is accomplished in some places where valves are not used, by plugging the end of the sewer with a disk consisting of sheet rubber faced with canvas and held firmly between boards about 1/2 in. smaller than the diameter of the sewer. When the tank is filled with water this plug is drawn out, thus starting the flush.

Automatic Flush-tanks.—The flushing done with automatic apparatus is generally much more frequent than where hand-flushing is practiced, the usual rule being to discharge the flush-tank once every 24 hours. The water is usually admitted to these tanks through special

orifices, of which a variety are manufactured by the makers of flushing siphons, so that any desired rate of flow under any street main pressure can be attained by screwing the proper orifice or jet into the end of the service pipe. As a rule these jets are also accompanied by a mud drum

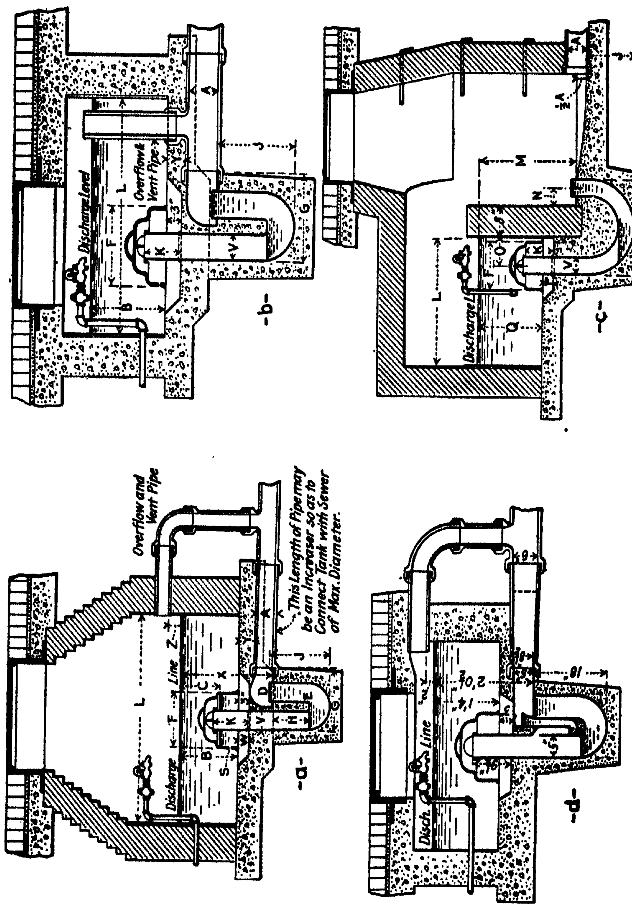


FIG. 259.—Different types of flush-tanks (Miller).

or screening device and a blow-off cock, provided to insure the jet against clogging.

The operation of a siphon of the simplest type is as follows: In Fig. 259*a*, the Miller siphon is shown just ready to discharge. There are two volumes of water separated by the compressed air in the long

leg V of the trap. As the pressure on every part of this confined mass of air must be equal to the hydrostatic pressure, and as there are but two places where water is in contact with the air, it follows that the depth of water C in the tank must be the same as the depth H in the trap. When the depth C is increased the water flows over the raised lip of the trap at D, this discharge allowing a little air to escape below the bend at E. The air pressure being released in this way, water passes up with a rush within the bell and into the long leg of the trap.

The elevation of the lip of the short leg at D above the bottom of the outlet is an important detail, as upon it the first sudden discharge of the trap seems to depend. In the older types of flushing apparatus this first strong flush was accomplished by using an auxiliary siphon at the bottom of the trap casting, a detail retained in the Rhoads-Miller siphon, Fig. 259*d*, for use where shallow construction is imperative.

When the water in the tank has been drawn down until its surface is below the snift hole S, air rushes into the bell and stops the siphonic action there. In consequence the water in the two legs of the trap at once forms a seal there and the apparatus is ready for discharge when the tank is filled again.

The dimensions of the Miller apparatus, required by designers, are given in Tables 161, 162, and 163. The diameter of the tank is the minimum which is generally considered desirable for siphons of the sizes listed. The discharge is the average given by the makers for that size and setting of siphon.

The setting shown in Fig. 259a does not afford access to the sewer, so the late Andrew Rosewater devised the special design shown in Fig. 259c to overcome this defect. The manhole at the dead end of the sewer is provided with a flush tank and siphon, and while this is more expensive than the standard type, it not only affords an opportunity to insert a cleaning rod into the end of the sewer, but is also stated to give a higher rate of discharge.

The same object, affording access to the end of the sewer is attained by placing the trap at right angles to the line of the sewer, instead of in the same line. This was first suggested by William Mackintosh. By locating the bell of the siphon at one side of the line of the sewer, the latter can be made to end in a special casting which not only receives the flush from the trap in the usual way but gives access to the sewer through a removable cover. This design, like the other Miller types, is made by the Pacific Flush Tank Co., and the sizes of manholes and dimensions and capacities of the siphons are the same as given in Table 161, for standard settings.

In the operation of these devices, the air needed to lock the apparatus during the filling of the tank enters the bell through a snift hole. If enough air is not admitted, the water may dribble continuously through

TABLE 161.—DIMENSIONS AND CAPACITIES OF MILLER SIPHONS, STANDARD SETTING, FIG. 259a

Sewer diam., in.	Discharge rate, cu. ft. per sec.	Bell diam., in.	Trap width, in.	Trap depth, in.	Trap rise, in.	Tank diam., ft.	Depth. in., from discharge line to		Floor depth, in.	Rise to overflow. in.	Trap diam., in.
							Floor	Invert			
A		F	G	J	K	L	B	X	Y	Z	V
4 to 6	0.35	$13\frac{1}{4}$	14	$14\frac{1}{4}$	$8\frac{3}{4}$	3	14	$22\frac{1}{4}$	4	3	4
6 to 8	0.73	$16\frac{3}{8}$	$18\frac{1}{4}$	$22\frac{3}{4}$	$9\frac{1}{2}$	3	23	34	5	2	5
8 to 10	1.06	$20\frac{1}{2}$	$20\frac{3}{8}$	$29\frac{5}{8}$	11	4	30	44	6	2	6
12 to 15	2.12	$25\frac{1}{2}$	$27\frac{1}{2}$	$36\frac{1}{2}$	$13\frac{1}{2}$	4	35	$51\frac{1}{4}$	$6\frac{1}{4}$	2	8

TABLE 162.—DIMENSIONS AND CAPACITIES OF MILLER SIPHONS, SHALLOW SETTING, FIG. 259b

A, in.	Discharge, c.f.s.	F, in.	G, in.	J, in.	K, in.	L, in.	B, in.	X, in.	Y, in.	Z, in.	V, in.
6 to 8	0.55	$16\frac{3}{8}$	$18\frac{1}{4}$	$17\frac{1}{4}$	$7\frac{3}{4}$	3	15	26	5	2	5
8 to 10	0.90	$20\frac{1}{2}$	20	$19\frac{1}{4}$	8	4	18	31	5	2	6

TABLE 163.—DIMENSIONS AND CAPACITIES OF MILLER SIPHONS, SPECIAL SETTINGS, FIG. 259c

A, in.	Discharge, c f.s.	F, in.	G, in.	J, in.	K, in.	L, ft.	M, in.	N, in.	O, in.	P, in.	Q, in.	V, in.
4–6	0.65	$13\frac{1}{4}$	$26\frac{1}{8}$	$13\frac{1}{4}$	9	3	29	5	8	3	19	4
6–8	1.02	$16\frac{3}{8}$	$30\frac{1}{8}$	$17\frac{3}{4}$	$9\frac{1}{2}$	3	$33\frac{1}{2}$	6	10	3	23	5
8–10	1.49	$20\frac{1}{2}$	$33\frac{1}{8}$	$24\frac{3}{4}$	11	4	43	6	12	4	31	6
12–15	2.97	$25\frac{1}{2}$	$39\frac{1}{4}$	$28\frac{3}{8}$	$13\frac{1}{2}$	5	$49\frac{1}{2}$	7	15	5	35	8

leg *V* of the trap. As the pressure on every part of this confined mass of air must be equal to the hydrostatic pressure, and as there are but two places where water is in contact with the air, it follows that the depth of water *C* in the tank must be the same as the depth *H* in the trap. When the depth *C* is increased the water flows over the raised lip of the trap at *D*, this discharge allowing a little air to escape below the bend at *E*. The air pressure being released in this way, water passes up with a rush within the bell and into the long leg of the trap.

The elevation of the lip of the short leg at *D* above the bottom of the outlet is an important detail, as upon it the first sudden discharge of the trap seems to depend. In the older types of flushing apparatus this first strong flush was accomplished by using an auxiliary siphon at the bottom of the trap casting, a detail retained in the Rhoads-Miller siphon, Fig. 259*d*, for use where shallow construction is imperative.

When the water in the tank has been drawn down until its surface is below the snift hole *S*, air rushes into the bell and stops the siphonic action there. In consequence the water in the two legs of the trap at once forms a seal there and the apparatus is ready for discharge when the tank is filled again.

The dimensions of the Miller apparatus, required by designers, are given in Tables 161, 162, and 163. The diameter of the tank is the minimum which is generally considered desirable for siphons of the sizes listed. The discharge is the average given by the makers for that size and setting of siphon.

The setting shown in Fig. 259a does not afford access to the sewer, so the late Andrew Rosewater devised the special design shown in Fig. 259c to overcome this defect. The manhole at the dead end of the sewer is provided with a flush tank and siphon, and while this is more expensive than the standard type, it not only affords an opportunity to insert a cleaning rod into the end of the sewer, but is also stated to give a higher rate of discharge.

The same object, affording access to the end of the sewer is attained by placing the trap at right angles to the line of the sewer, instead of in the same line. This was first suggested by William Mackintosh. By locating the bell of the siphon at one side of the line of the sewer, the latter can be made to end in a special casting which not only receives the flush from the trap in the usual way but gives access to the sewer through a removable cover. This design, like the other Miller types, is made by the Pacific Flush Tank Co., and the sizes of manholes and dimensions and capacities of the siphons are the same as given in Table 161, for standard settings.

In the operation of these devices, the air needed to lock the apparatus during the filling of the tank enters the bell through a snift hole. If enough air is not admitted, the water may dribble continuously through

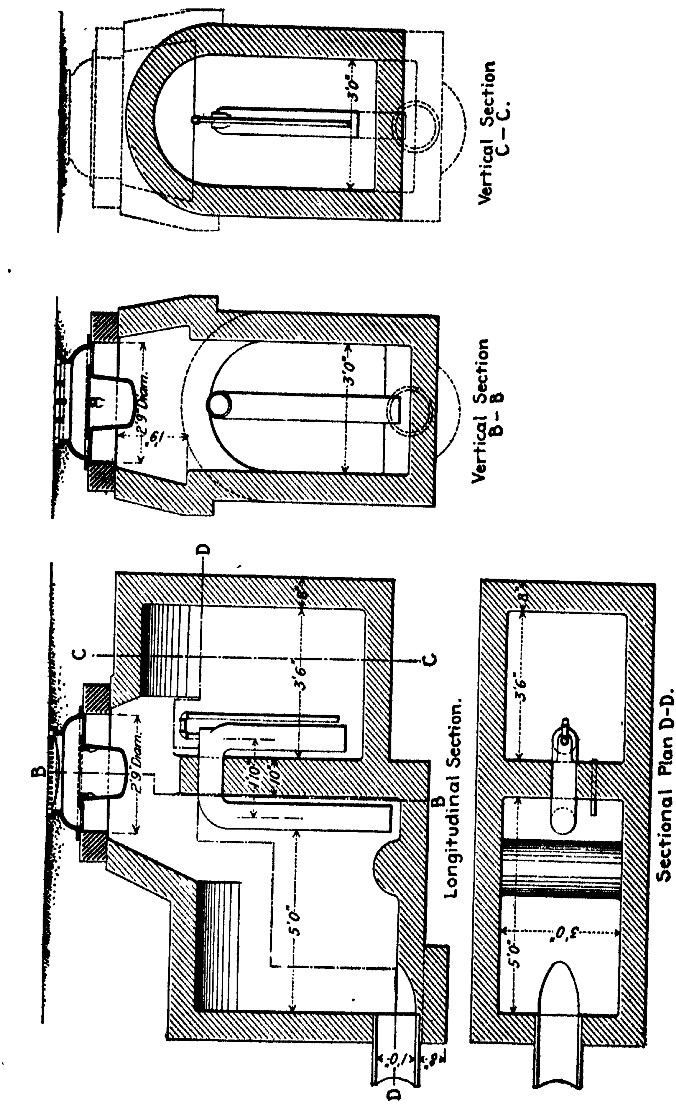
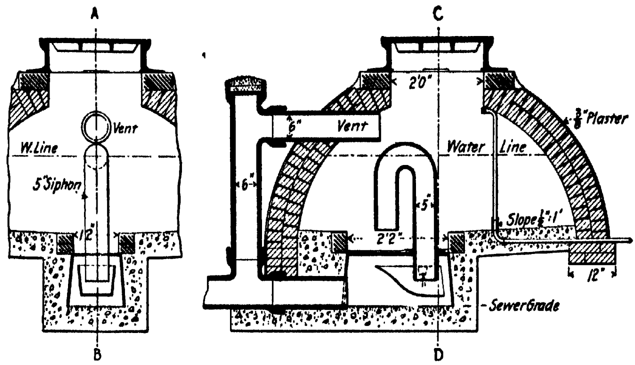


FIG. 261.—Standard flush-tank, Winnipeg.

its gradient. From the general consideration of the well-known formula for velocity, $V = C\sqrt{RS}$, remembering that for circular conduits the hydraulic radius is a direct function of the diameter, we may consider (1) the quantity Q varies directly as the square root of the radius and inversely as the square root of the slope, and, to complete the statement of controlling conditions, (2) that it varies directly as the length from the dead end to the point where the normal flow becomes sufficient to maintain a velocity of 2-1/2 ft. per second. Under these assumed conditions, designating this distance by L, letting c represent the necessary modifying coefficient, the formula would take the shape, $Q = L\sqrt{R} \div c\sqrt{S}$.

"Solving this equation for the data given on the Park Street line (given in the opening of the paper) we obtain a rough approximate for c of 190.

"Let us now consider the factors which establish the value of L. If we



Section C-D. Section A-B.

FIG. 262.—Van Vranken flush-tank.

let A represent the area of the cross-section of normal flow for any given gradient required to produce the velocity of 2-1/2 ft. per second, and let D equal the increment in discharge for each linear foot of sewer in cubic feet per second, then $L = 2.5A \div D$, in which A is definitely determined by an application of Kutter's formula. The quantity D is evidently a function of the number of persons or premises tributary to the sewer and of their *per diem* water consumption. But these are variable quantities, rarely the same for two sewers. A uniform contributing population of 30 persons per 100 ft. of sewer with a daily flow per capita of 100 gal., three-fourths assumed to run off in 6 hours, would give a value for D of 0.00015 cu. ft. per second. Table 164 gives the value of A for different grades, and the corresponding depth of flow in inches. This table indicates the very small flow on the larger grades necessary to maintain a self-cleansing velocity, and the relation between the ordinary discharge and the grade within the limits given.

"Table 165 gives the various quantities of water given by the formula for the foregoing grades and sizes under the conditions which have been

stated, allowing an increased rate on long lines and a diminishing rate of flow on short lines form the average value of 0.00015 cu. ft. per second.

"These results indicate that a very considerable modification of the volume of water should be allowed for lines of different gradient, and that the required volume diminishes very rapidly with an increase of grade; also that it is affected to a smaller extent by the size of the sewer, that for all sizes no flush-tanks are probably required on slopes exceding 2 per cent., and it may be inferred in such cases, also, that flushing at less frequent intervals is needed than the 24- to 48-hour discharge.

TABLE 164.—SECTIONS IN SQUARE FEET AND DEPTHS IN INCHES TO PRODUCE A VELOCITY OF 2½ FEET IN SEWERS 6 TO 12 IN. IN DIAMETER AND ON GRADES OF 0.5 TO 5 PER CENT.

Grade		Diameter of sewers 6 in.	8 in.	10 in.	12 in.
½	Area	....	0.229	0.226	0.237
	Depth	....	5.0	4.3	4.1
¾	Area	0.125	0.130	0.137	0.150
	Depth	3.9	3.2	3.0	2.9
1	Area	0.095	0.101	0.108	0.115
	Depth	2.9	2.7	2.5	2.4
2	Area	0.043	0.050	0.55	0.060
	Depth	1.7	1.6	1.6	1.5
3	Area	0.031	0.035	0.037	0.041
	Depth	1.3	1.3	1.2	1.2
4	Area	0.022	0.025	0.028	0.031
	Depth	1.0	1.0	1.0	1.0
5	Area	0.017	0.021	0.024	0.027
	Depth	0.9	0.9	0.9	0.9

TABLE 165.—GALLONS OF WATER REQUIRED FOR FLUSHING

Grade, per cent.	Diameter of sewers 8 in.	10 in.	12 in.
½	80	90	100
¾	55	65	80
1	45	55	70
2	20	30	35
3	15	20	24
4	10	15	20
5	8	10	15

An investigation of the action of water in flushing sewers was made by Prof. H. N. Ogden at Ithaca, N. Y., about 1898, and the results are described by him in a paper in *Trans. Am. Soc. C. E.*, vol. xl, page 1. This investigation was begun to determine the necessity of a flush-tank at the end of every lateral sewer in that city, in accordance with a recommendation made by the designer of the system. Professor Ogden's correspondence with other engineers showed a wide diversity of

opinion, some preferring hand flushing, others automatic flushing, and still others combinations of the two. A few had taken up hand flushing because of disastrous experience with automatic apparatus, and a few had adopted flush tanks because they found it impracticable to obtain good hand flushing.[1] Little practical information was apparently obtained, although one engineer reported that experience on the sewer system under his charge indicated that one flush daily on a 2 per cent. grade was as effective as two flushes daily on a 0.5 per cent. grade, each flush being of 300 gal. The general opinion was that occasional flushing was needed on the upper ends of all laterals on grades below 1 per cent.

Professor Ogden's experiments were made on 8-in. pipe sewers, each with a 4-ft. manhole at its upper end. The end of the sewer was stopped with a pine board having a 5-in. orifice, closed by a rubber-faced cover. The manhole was filled with water to depths of 4 to 6 ft. and when the cover was removed the water was discharged at rates of 0.89 to 1.1 sec.-ft. The depth of this discharge and its effect in moving gravel were observed at successive manholes down the sewer. Flushes of 20, 30, 40, 50 and 60 cu. ft. were used successively.

As a result of these investigations Professor Ogden reached the conclusion that the volume of water discharged should not be less than 40 cu. ft., and the effect of the flush can hardly be expected to reach more than 600 or 800 ft. If tanks are used on grades greater than 1 per cent., 15 to 20 cu. ft. give as good results as larger amounts, but on such grades hand-flushing will be more economical than automatic flushing.

In inquiries concerning the capacity of flush-tanks a definite rule was received only from the Van Vranken Flush-Tank Co., which stated that the capacity of the tank should be equal to half that of a length of sewer in which the grade produces a rise equal to the diameter of the pipe. It was the opinion of the manager of the Pacific Flush-Tank Co. that a flush of 175 gal. on a 1 per cent. grade was sufficient, and on flatter grades twice that quantity of water should be used.

In the discussion of this paper, George W. Tillson stated that in Omaha on 6-in. lateral sewers with grades of 1/2 to 8 per cent. and no flush-tanks, a growth of fungus half filled the bore of the laterals in the course of a year or two. In later work of the same sort, flush-tanks discharging every 12 hours were used at the dead ends of the laterals, and no trouble from the fungus was observed in such cases.

[1] George G. Earl, superintendent of the New Orleans Sewaerge and Water Works, informed the authors in 1913 that while there are automatic flush-tanks on all dead ends of the sewers in that city, they are not operated constantly. "Instead, we have two men constantly going over the system, covering all flush-tanks about twice a month and giving four or five flushes in rapid succession just as fast as a 1-in. pipe and gate valve, direct connected to the flush-tank, can fill them. This makes wave follow wave down the sewer, and we think saves water and gets better effect in flushing and reaches further from the flush-tank with an effective flush than two or three automatic discharges per day each. In addition to this we keep two gangs going over all sewers constantly with ball and flush cleaning."

CHAPTER XVI

REGULATORS, OVERFLOWS, OUTLETS, TIDE GATES AND VENTILATION

The function of a sewage flow regulator is to prevent the surcharge of an intercepting sewer, by closing an automatic gate upon the branch sewer connection, thus cutting off the sewage and forcing it to flow to another outfall.

A storm overflow is designed to allow the excess sewage above a definite quantity to escape from the sewer in which it is flowing through an opening.

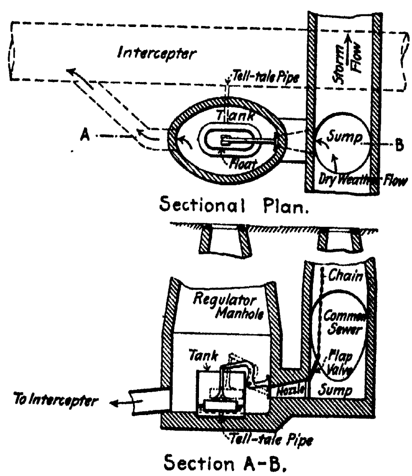


FIG. 263.—Old type of Boston regulator.

The purpose of both devices is substantially the same, namely, to allow the ordinary flow of sewage to be delivered to a distant point of discharge, and at the same time to cause the excess storm flow, which is very much less foul, to be discharged into the nearest watercourse. Sometimes regulators are used in combination with storm overflows to safeguard an intercepting sewer by cutting off entirely the sewage entering the intercepter when the latter is filled to a certain point. The

overflow allows the escape of excess storm flow; the regulator finally causes the entire flow in the branch sewer, both sewage and storm water, to pass the overflow and be discharged into the nearest watercourse.

REGULATORS

A discharge regulator usually consists of an automatic gate operated by a float which rises or falls as the elevation of the sewage

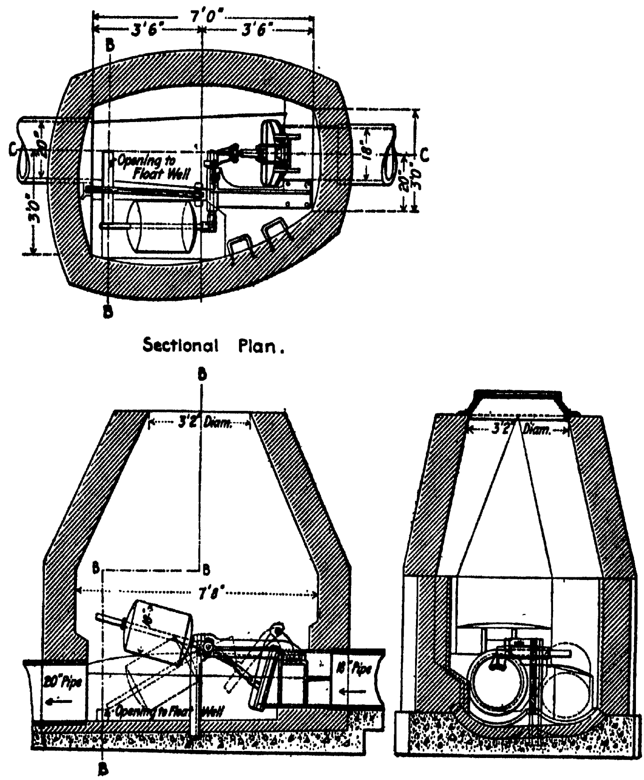


Fig. 264.—New type of Boston regulator.

increases or decreases. When the intercepter is filled to its capacity the gate closes entirely and further discharge of sewage into the intercepter is cut off.

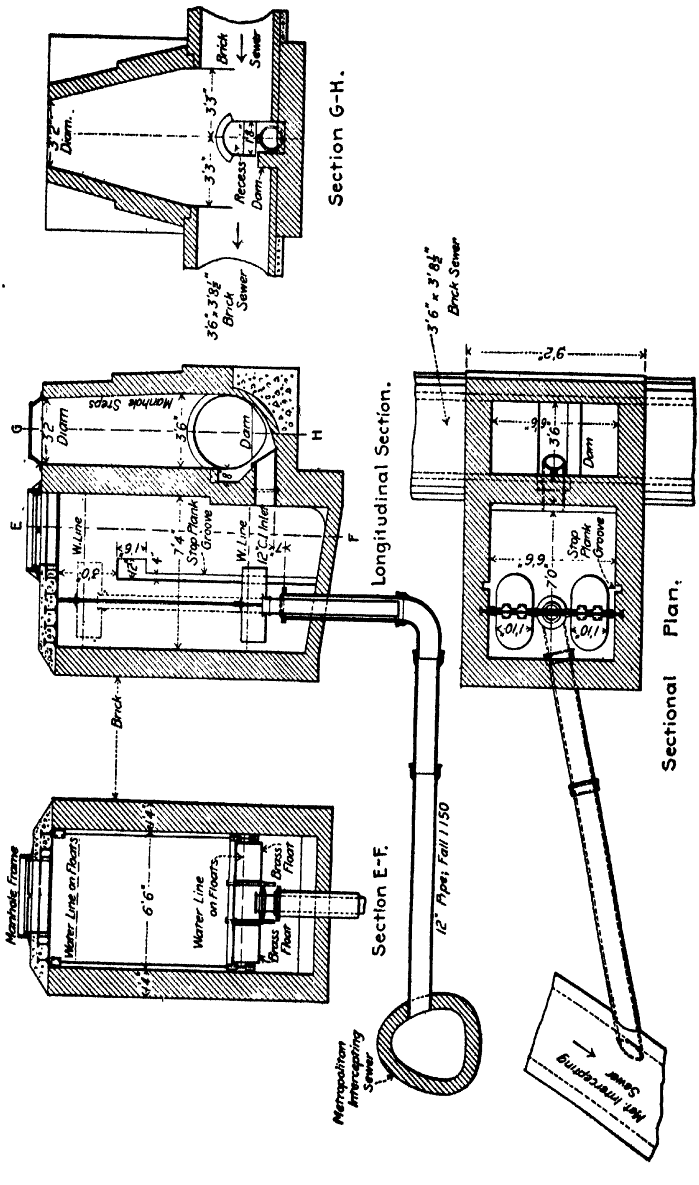


Fig. 265.—Constant flow regulator, Boston.

Probably the best-known type of regulator is shown in Fig. 263; this is used on the connections between the Boston main sewers and the Metropolitan intercepting sewers. The structure consists, in brief, of an orifice in the trunk sewer, a pipe connecting this orifice with the intercepting sewer, a regulating gate, a float to operate the gate automatically, and a telltale pipe through which the height of sewage in the intercepting sewer is communicated to the float chamber.

The orifice in the trunk sewer is designed of sufficient capacity to allow the proper quantity of sewage to pass through it. In some cases it is necessary to provide a low dam in the trunk sewer at a point immediately below the orifice to assist in diverting the sewage. The pipe leading from the orifice may pass through the regulating chamber and thence to the intercepting sewer. The regulating gate seats against a cast-iron nozzle which forms the orifice in the trunk sewer. This gate is carried on the end of a lever, to the other end of which is attached a

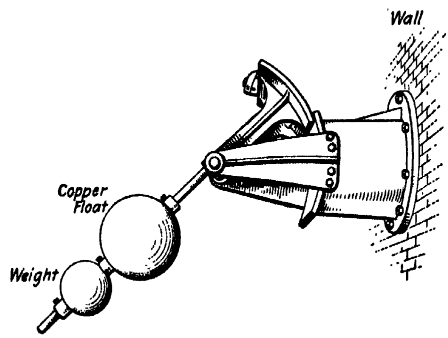


Fig. 266.—Coffin regulator.

large float which rises and falls in the float chamber with the rise and fall of sewage in the intercepting sewer, the communication of the height of sewage between the intercepting sewer and the float chamber being accomplished by means of a telltale pipe of small size which connects the two. Thus as the depth of sewage in the intercepting sewer increases in time of storm, the float is raised and correspondingly the gate is lowered or closed. When the intercepter is as nearly full as desired, the gate through which the sewage flows is closed, thus preventing the flow of more sewage into the intercepter, and at the same time causing the sewage and storm water to flow past the orifice through the lower part of the original trunk sewer into the river.

The experience with the mechanical features of these regulators has been satisfactory except in one respect. There has been a tendency in some installations toward the formation of deposits around the central

float chamber, and to avoid this a later arrangement, Fig. 264, was developed by C. H. Dodd under the direction of E. S. Dorr, chief engineer of the Boston Sewer Service. The toggle joint gives a leverage of about 16 : 1 when the valve closes, which is a help in reducing leakage. Only one float is needed for the regulators on 8 and 12-in. sewers.

Where it is desired to intercept only a constant volume of sewage, recourse may be had to a constant-flow regulator, of the type shown in Fig. 265. The depth of the sewage over the entrance to the vertical telescopic outlet pipe is maintained constant by lifting or lowering the pipe as the level of the sewage fluctuates. This motion is produced by the two large brass floats attached to the top of the pipe.

The simplest type of regulator is shown in Fig. 266, and is made by the Coffin Valve Co., of Boston. It has a cast-iron body which is bolted to the end of the branch sewer and projects into the intercepting sewer or a tank connected with it in which the sewage will rise to the same height as in the intercepter. The valve and frame are fitted with composition facings, hammered into dove-tailed grooves and pinned. The valve and its seat are machined and then scraped by hand to give a reasonably tight circumferential bearing. The steel shaft carries an adjustable copper float and a weight by which the action of the device can be somewhat varied. This regulator is also employed as a backwater valve to prevent sewage backing into branches from a main sewer that becomes surcharged.

Other types of regulators used at Syracuse, N. Y., are shown in Figs. 267 and 268, which require no comment. There is a limit, of course, beyond which it is hardly wise to expect such apparatus to operate automatically, and it is not surprising that one of these regulating valves refused to work according to the chief engineer and designer of the intercepting sewerage system, Glenn D. Holmes, after it had become clogged with a 2 × 10-in. plank 5 ft. long, a roller 6 in. in diameter and 4 ft. long used in moving buildings, a 2-ft. length of a similar roller, a 4 × 8-in. timber 4 ft. long, mop and handle, broken crockery, rags and small sticks. How such collections of large objects get into the sewers in the first place and are gathered at one spot after entering them, is one of the questions which occasionally puzzles the superintendent of any large sewerage system.

A type of regulator is used at Washington, D. C., in which the floats are operated by clean water from the city mains, admitted to the float chambers through valves controlled by the rise and fall of sewage back of an overfall dam. Asa E. Phillips, superintendent of sewers, stated in 1913 that the most elaborate installation, shown in Figs. 269 and 270, had then worked with absolute regularity for 2 years. It is so well balanced that it delivers the sewage from the trunk sewer into the 3-ft. intercepter so long as the latter is not filled. As soon as the full capacity

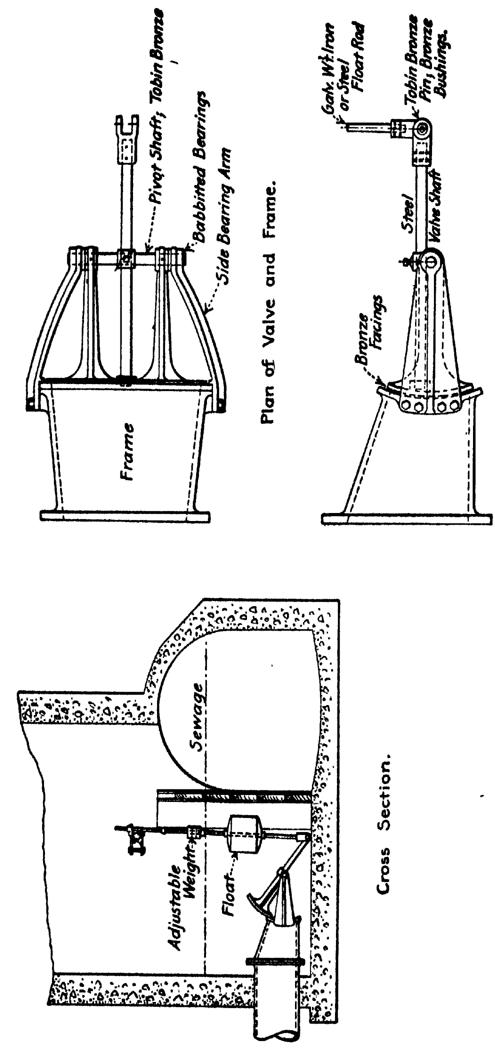


FIG. 267.—Regulator used at Syracuse, N. Y.

of the intercepter is being utilized, the regulator cuts off the flow to the intercepter, and as soon as the latter is able to receive more sewage, the regulator starts the flow again. The following description of its operation is from *Eng. Record*, vol. lxv, p. 312.

"The apparatus for controlling the quantity of storm flow delivered to the 3-ft. intercepter, and for cutting out excessive storms, is located in an underground concrete gate chamber built just off the main line, and con-

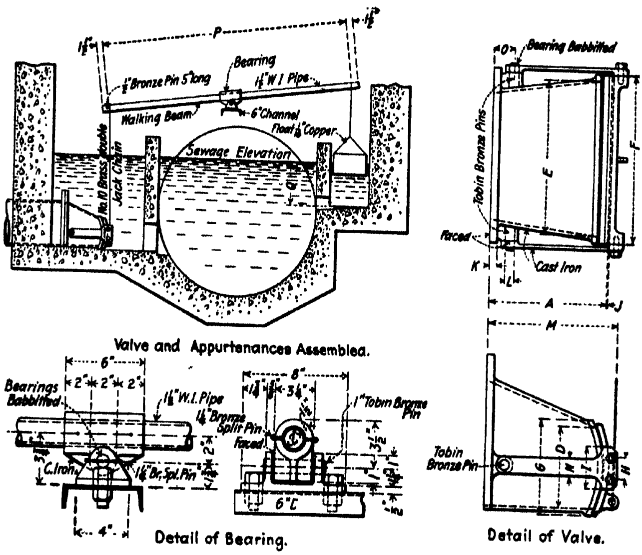


TABLE OF DIMENSIONS FOR VALVES

Size S	A	No.	B	C	D	E	F	G	H	I	J	K	L	M	N	O	Flange Bolts Diam.	Flange Bolts Length	Flange Bolts No.
8"	11¾"	10	13½"	11¾"	4¾"	11"	13"	6¾"	2¼"	4¾"	5/16"	¼"	1"	12¾"	2"	2¼"	¾"	2¾"	7
12"	14½"		19"	17"	7½"	15½"	17½"	9½"	2½"	5"	¼"	⅜"	1¼"	16⅛"	2¼"	2½"	⅞"	3¼"	10
18"	19¾"	2	25"	22⅝"	11⅝"	22"	24"	13⅝"	3"	6"	5/16"	1⅛"	1½"	21"	2½"	3"	1⅛"	4"	13
24"	24"		32"	29½"	16¼"	28"	30"	18¾"	5"	8"	5/16"	1⅛"	1½"	25⅜"	2½"	3"	1¼"	4¼"	16

FIG. 268.—Regulator used at Syracuse, N. Y

nected thereto by a 3-ft. conduit. Above this connection the trunk sewer is transformed in section from a circular to a cunette section, thus forming a collecting channel for the diversion of the flow to the gate chamber. This cunette extends as a tongue below the 3-ft. outlet conduit for the purpose of diverting from the intercepter the heavy material such as cobble and boulder, which excessive storms bring down from raw surface areas within this drainage district. Just where this tongue of the cunette dies cut in

the berme, a slight ridge is raised, forming a low cross dam for the purpose of holding the hydraulic gradient at such a level that the 3-ft. intercepter will run full before any discharge is spilled into the stream.

"The automatic regulating apparatus is designed to entirely shut off the flow from the intercepting sewer just as soon as the latter is running full. Under this condition the flow in the trunk sewer is about level with the top of the diverting weir. This result is accomplished by means of two valves with disks in the form of cylindrical surfaces, which slide upon bronze seats in castings imbedded in a concrete bulkhead wall across the line of flow. From these disks arms project with floating balls of copper on the ends of same and working in a pair of concrete tanks, so that by automatically filling and emptying the tanks at the proper time the balls are made to rise and fall and to close and open the valves.

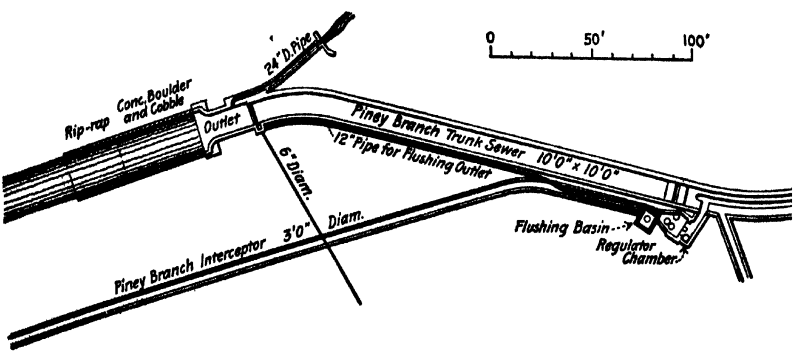


FIG. 269.—Plan of regulating works, Piney creek sewer, Washington.

"At the proper level below the diverting dam in the main sewer to give the required discharge, as checked by experiment in the shop, a small pipe is introduced and leads to a pair of outlets in the regulating chamber, each one directly over a small funnel pail, hung from a lever arm in such a way that a downward movement of the funnel lifts a ball valve on a 2-in. pipe outlet from a 10-ft. capacity reservoir suspended from the roof of the gate chamber, which is filled through a float-controlled valve by a pipe connection with the city water main. This 2-in. pipe discharges directly into the float tank below on the floor of the gate chamber, and raises the large copper float which closes the automatic gate.

"When the flow in the main sewer rises above the inlet pipe just below the level of the diverting weir, water passes into the two suspended funnels, filling them and thereby causing sufficient weight on the end of the lever arm to lift the ball valve on the outlet from the reservoir, which is connected to the float tank. The water rapidly rises in the latter, lifting the copper float and gradually closing the segmental slide-valve in the bulkhead wall. This shuts off the flow of sewage into the 3-ft. intercepter and automatically

diverts same to Piney Branch. Peak load of the storm is thus entirely discharged into the stream. But as soon as the run-off is sufficiently reduced, the controlling gates open and the flow is once again diverted to the intercepting sewer. The operation is as follows:

"When the flow in the main sewer drops to the capacity of the 3-ft. diameter intercepter it is just level with the inlet to the small pipes leading to the funnels, so that the flow which has kept the latter full is reduced below the discharge capacity of the funnel outlets, and the water therein quickly drains away, reducing the pull on the lever arm from which they are suspended, and thereby causing a counterweight on the extension of the arm to close the ball valve feeding the float tank.

"These float tanks are drained by small outlet holes and when the feed supply is thus cut off they slowly empty and the floats descend, gradually opening the sliding valves, and delivering the discharge to the 3-ft. intercepter. This condition continues until the next excessive storm discharge. Meantime the reservoirs over the float tanks have filled from the city water supply, and are ready for the next storm.

"During the period when gates are closed, the city water continues to flow into the reservoirs, and thence into the float tanks, thus keeping up the floats which hold the controlling gates shut, notwithstanding the small outlet holes which are always open and continue during this period to waste, the inflow, of course, being set to exceed this outflow. This is accomplished with a 1/2-in. supply pipe. The feed pipe leading from the main sewer to the funnels is protected by a screen and so connected and valved in the gate chamber that the city water pressure may be turned through same for flushing out the pipe and cleaning the screens. This connection also serves to permit the testing out of the apparatus at any time. Immediately after storms it is the practice to have an inspector visit the works to examine same and do any special flushing necessary."

Another type of installation in Washington is shown in Fig. 271. The regulators are for the purpose of shutting off the intercepting sewer completely at this place, when this becomes necessary, and diverting the sewage from the 6-ft. sewer heading to the pumping station over the sill of a relief outlet, or into a bypass, leading to a 6-ft. storm-water relief conduit running to the Potomac river. Below the elevation at which this regulating structure operates all storm water has to be pumped, as this is the lowest place from which there is a gravity discharge. The regulators are operated by the same kind of apparatus described in connection with the first Washington installation. In November, 1913, Mr. Phillips wrote to the authors as follows: "We have 14 regulator chambers of this general character at present in the system, and some half dozen additional planned for construction. All those in service have given most satisfactory results with the float-tank construction noted above. We have never attempted the hazardous experiment of placing the float directly in the sewer to be actuated by changes in level of the sewage flow itself."

In Rochester, N. Y., where sewers are built in tunnels as shown in Fig. 272, City Engineer E. A. Fisher has adopted the type of regulator shown in that illustration. This has unusually sturdy members in proportion to the 12 × 20 in. opening which is under control, and is also unusual in that the disk is not designed to be able to shut off the discharge opening completely. This closing can be accomplished by hand, however. The operation of this regulator is described as follows in the report (1913) of Mr. Fisher on the sewage disposal system of Rochester:

"It is contemplated taking into the intercepting sewer all of the sewage and two and one-half additional volumes in time of storm. The storm water in the outlet sewers in excess of this quantity will pass on and discharge into the river, the existing sewers thus becoming overflows beyond the point of interception. In order to control the flow to be diverted into the intercepter, chambers will be constructed in which regulating devices will be installed that will automatically maintain the required volume of discharge. These regulating devices will be operated by a float, located in a chamber in which the water will rise and fall as the volume entering the chamber is in excess of, or less than, the volume discharging. As the water rises the float will operate a shutter closing the inlet, thereby reducing the volume entering until it is equal to the volume discharging; or if it grows less than the volume discharging, the water in the chamber will naturally fall, thereby causing the float to again open the shutter. The discharge from the chamber is fixed by the size of the opening and a given head. In each case the regulating device must be adjusted so that the float will begin to operate by closing the shutter when this given head is reached. In order to provide for a larger discharge, as the amount of sewage increases from year to year, the size of the opening from the chamber will be enlarged in order to give the area required with the given head to produce the discharge desired."

A special regulator has been constructed by George A. Carpenter, City Engineer of Pawtucket, R. I., using a gate valve operated by a hydraulic plunger, controlled by the old type of Venturi meter recording apparatus, actuated by a float. In this case it was desirable to have the entire dry weather flow and the first wash of the streets at times of storms taken to the treatment works, and to turn the entire flow of the sewer into the nearest water-course when the dilution reached a certain point, reversing the operation when the total flow fell below another predetermined amount, less than that for which the gate was closed. The sewage flows through an orifice in the bottom of the diversion chamber into a pipe upon which the hydraulic valve is established. A float in the diversion chamber moves a vertical rod upon which are tappets, one of which controls the opening and the other the closing, of the hydraulic valve. When the quantity reaches that for which the valve should be closed, the tappet trips the Venturi register apparatus, which thereupon operates a small valve admitting water from the city water works to the hydraulic cylinder and closing the valve. When the flow again falls to

the point at which the valve should be opened, the other tappet trips the mechanism to reverse the valves and open the main valve. Since the only power required from the mechanism is that consumed in opening and closing the small valves in the pressure pipes, it has been found that one winding of the weights of the Venturi recording apparatus is sufficient for more than 200 operations of the hydraulic valve. A description of the valve will be found in the *Journal of the Boston Society of Civil Engineers*, October 1914.

At Cleveland, Ohio, where regulating valves of the walking-beam type were tried unsuccessfully, the gate which was operated by the float was of the sluice class, rather than the curved class generally employed. The gate frame was made of cast iron and provided with a phosphor bronze seat; the gate was cast iron. The main bearings of the walking-beam had bronze bushings and attention had been paid in the design to the elimination of friction and opportunity for any binding of the parts. The following note on the failure of this regulator has been furnished by J. M. Estep, Assistant Chief Engineer of the Department of Public Service of the city: "The trouble with this type of regulator has been that the sliding gate which shuts off the flow at a certain elevation of the storm water in the chamber, fails to operate properly in the phosphor-bronze slides, and I think the gate probably remains open so that this type of overflow acts just as the ordinary overflow where a diversion dam is used."

The construction of automatic regulators and the nature of the sewage and water passing through them are such that frequent inspection is necessary to assure their effective operation. Regulators and tide gates should be inspected every day, and immediately following storms the cleaning and inspecting force should be increased so that all regulators which are clogged can be put into working condition at the earliest possible minute. It is only by this means that automatic regulation will be satisfactory.

OVERFLOWS

Storm overflows are of two types, overfall and leaping weirs.

An overfall weir is usually constructed in the side of a sewer, and the excess flow escapes over the crest when the elevation of the sewage is above that of the weir. One method of design of such a structure is described fully by W. C. Parmley, in a paper on the Walworth Run sewer in Cleveland in *Trans. Am. Soc. C. E.*, vol. lv, p. 341.

The main sewer entering the relief chamber is 14 ft. 9 in. in diameter, with a maximum calculated flow of 2500 cu. ft. per second. The calculated maximum flow of dry-weather sewage is about 30 cu. ft. per second. The intercepting sewers are designed to carry the dry-weather flow and an equal volume of storm water, in order to provide for the interception

of the foul, first flow from the streets. The required carrying capacity of the intercepting sewer at this point, therefore, is about 60 cu. ft. per second. The problem was to design a structure which would always divert 60 cu. ft. per second before any storm water was discharged to the main outlet, and one also that would not divert more than this amount under any condition of storm flow in the main sewer. Mr. Parmley's solution of the problem is as follows:

Suppose one side of the sewer to be cut away and converted into an overflow weir such that the flow of the volume of water below the level of this weir is not obstructed, but that all the water above its level can discharge sideways over the weir. With a given depth upon the upper end of the weir the water will tend to be discharged sideways according to the ordinary weir formula. There is, however, the forward velocity of the water in the sewer behind the weir to be considered. In the first unit of length a given quantity of water per second will be discharged, thereby reducing the head upon the weir in the second unit of length; this reduction of head in the second unit of length, caused by the water discharged in the first unit of length, will make the rate of overflow in the second unit less per second than it was in the first. In a similar manner each succeeding unit of length of weir will discharge a less volume than the preceding unit, owing to the continual reduction of head as the water moves forward in the sewer. The forward velocity in the sewer tends also to slacken, due to the lessening volume carried. An analysis of the problem shows that, theoretically, a weir would have to be of infinite length in order to reduce the water to the level of the crest of the weir; therefore it is not attempted to discharge all the water above the level of the weir, but to reduce the head upon the weir to some small amount. The problem involved may be stated as follows:

Let Fig. 273 represent the cross-section of the overflow chamber at the upper end of the weir, at the point where the water emerges from the sewer.

Let X and Y represent the axes of co-ordinates, with the origin in the axis of the sewer. Consider this section to represent a unit length of sewer.

Let A be the crest of the weir, and let $a + y$ be the depth of water over the weir.

Let the radius of the sewer equal r.

The co-ordinates of the weir, therefore, are $x = x_1$ and $y = -a$.

How long will it require for the water flowing over the weir to reduce the head of water on the weir from $a + y$ to any given lesser head?

Let dQ equal the volume of water discharged for a reduction of head, dy, and let dt equal the time required for the discharge of the quantity, dQ.

We then have the equations:

$$dQ = 2xdy = 2\sqrt{(r^2 - y^2)}dy$$

For the head $a + y$, the rate of discharge, q, equals approximately $3.33\ (a + y)^{3/2}$

then

$$dQ = qdt = 3.33(a+y)^{3/2}dt,$$

therefore,

$$dt = \frac{0.6\sqrt{(r^2 - y^2)}}{(a+y)^{3/2}}dy.$$

Integrating between the limits, y_1 and y_2, for any two heads upon the weir, gives the time required to reduce the head from y_1 to y_2. It has not been possible, however, to integrate this equation, and, therefore, it has been necessary to mak? use of it in the approximate form:

$$t = \sum\left[\frac{0.6\sqrt{(r^2 - y^2)}}{(a+y)^{3/2}}\Delta y\right]_{y_2}^{y_1}$$

Obtaining the Δt for successive differences in head, Δy, between the limits y_1 and y_2, and taking the sum of all these Δt's will give the approximate time, t, required.

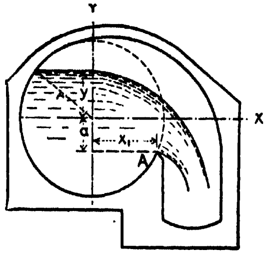


FIG. 273.

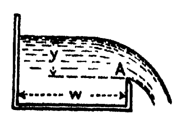


FIG. 274.

This being a tedious process, an approximation can be made by reducing the circular sewer to a rectangular one of the same average width. In this case, let Fig. 274 represent the cross-section of the rectangular sewer, with the weir at A, and with an initial depth of water, y, over the weir. Let the width of channel, w, equal the average width of the circular sewer, shown in Fig. 273, to the left of the weir, A. In this case the water overhanging the weir on the right is assumed to fall away by the force of gravity without interfering with the weir discharge of the water over and back of the weir. In this case, then, we have

q = the rate of discharge for the head $y = 3.33y^{3/2}$:

and

Q = the total quantity discharged.

For an infinitesimal reduction in head, dy, we have

$$dQ = wdy = qdt = 3.33y^{3/2}dt$$

therefore

$$dt = \frac{w}{3.33}y^{-3/2}dy$$

Integrating between the limiting heads, y_1 and y_2, gives

$$t = \left(-\frac{w}{1.67\sqrt{y}}\right)_{y_2}^{y_1} = \frac{w}{1.67}\left(\frac{1}{\sqrt{y_2}} - \frac{1}{\sqrt{y_1}}\right)$$

If $y_2 = 0, t = \infty$, which shows, as before stated, that theoretically it would require a weir of infinite length to reduce the water to a zero head. The last formula is simply and easily applied, and does not give results varying greatly from those obtained from the differential equation for the circular sewer.

If the velocity in the sewer were constant while flowing the length of the weir, and if all the filaments in the entire cross-section had the same velocity, the foregoing equation would give the time required to reduce

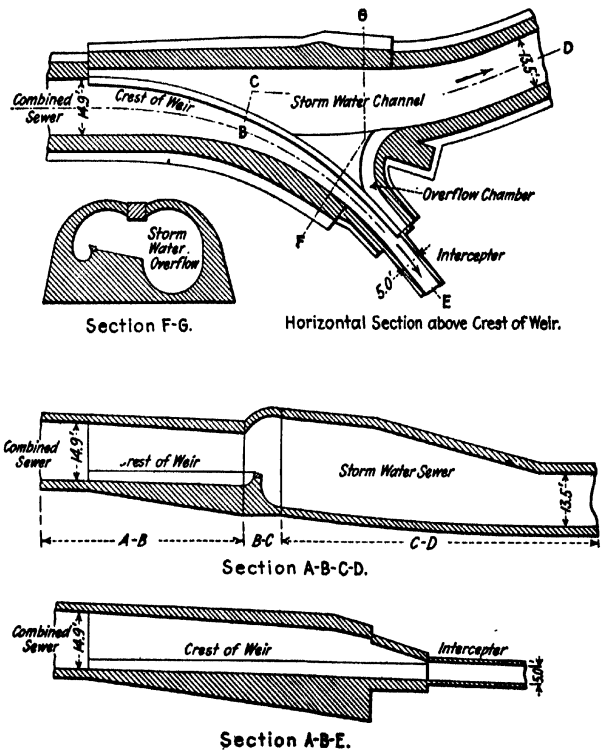


FIG. 275.—Weir for storm-water overflow, Cleveland.

the level of the water from one stage to another, and this time multiplied by the velocity of flow in the sewer behind the weir would give the length of weir required. These ideal conditions, however, are not obtained in practice. The velocity in the sewer is gradually retarded as the head becomes less, and, consequently, the sill must be lengthened somewhat in order to perform the same amount of work.

By referring to the completed design, Fig. 275, it will be seen that the dry-weather channel was built upon a curve gradually decreasing in width and size as it passes from the 14-ft. 9-in. sewer to the 5-ft. intercepter.

In order to avoid backwater for partial depths of flow, a number of calculations were made with varying depths of flow in the main sewer. As a result of a large number of these calculations for assumed relative elevations of inverts of main and intercepting sewers and of varying volumes of flow, the following conclusions for regulating the design of the overflow chamber were obtained:

Since it was not desirable to allow the velocity in the main sewer above the overflow chamber to be reduced below about 2.50 ft. per second, it was necessary to make a drop of at least 1.50 ft. in passing from the invert of the 14-ft. 9-in. sewer to the invert of the 5-ft. sewer.[1] With this drop, the minimum velocity in the main sewer will be about 2.50 ft. per second, when 60 cu. ft. per second are flowing. For a less quantity than 60 cu. ft. per second, there will be an acceleration in the velocity above the junction for any small volumes of flow, and for no quantity less than 60 cu. ft. per second will the effect of backwater reduce the velocity to less than 2.50 ft. per second. For volumes greater than 60 cu. ft. per second, the sill of the overflow must be long enough[2] to take out all but 60 cu. ft. per second, which will remain in the sewer to be carried off by the intercepter. For the maximum discharge of 2500 cu. ft. per second for a 14-ft. 9-in. sewer there will be no backwater effect. Hence the 14-ft. 9-in. sewer will flow unobstructed when nine-tenths full.

The elevation of the upstream end of the weir, therefore, was placed 2.70 ft. above the invert of the 14-ft. 9-in. sewer, and is carried to an elevation of 4.50 ft. above the invert of the 5-ft. intercepter after the invert of the intercepter has been fixed at a proper elevation as above determined. The grade of the crest of the weir is 0.3 ft. per 100 ft. The form of cross-section of the dry-weather channel at the upper end begins as the segment of the 14-ft. 9-in. sewer, and, in passing downstream to the 5-ft. intercepter, gradually changes to the section of the 5-ft. sewer with the crest of the overflow sill nine-tenths of the diameter of the sewer above the invert.

In order to avoid any backwater effect from the storm water overflow, it is necessary that the weir should never act as a submerged weir. That is to say, the surface of the storm water in the overflow channel must always be lower than the crest of the weir. The storm-water branch below the overflow chamber was given a drop of about 12 ft. below the level of the sill, and was carried down the valley on a grade of 0.50 ft. per 100 ft. The overflow branch, therefore, was made 13 ft. 6 in. in diameter.

The standard type of overflow manhole used on the larger sewers in Cleveland, Ohio, is shown in Fig. 276. Attention is called to the fact that the grade for the dry-weather flow in this case is about 0.46 ft. in 4-1/2 ft.

[1] Depth of flow in 5-ft. sewer would be about 4.5 ft.; in 14-ft. 9-in. sewer about 2.7 ft. The difference, 1.8 ft., would be drop in invert if water surface were level.

[2] The length of the weir was made about 95 ft.

Concerning the intercepting chambers built at Syracuse and shown in Fig. 277, Chief Eng. Glenn D. Holmes of the Intercepting Sewer Board makes the following statements: "No weir is used in this type

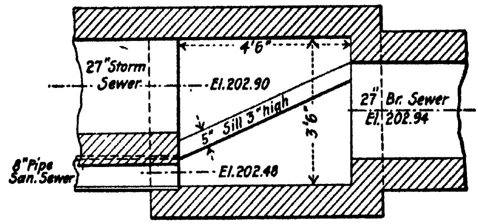


FIG. 276.—Overflow manhole, Cleveland.

but a small dam is placed in the old sewer just beyond the chamber. The dam is usually built with flat slopes on the upstream and downstrean sides to give an effect similar to that of the throat of a Venturi meter.

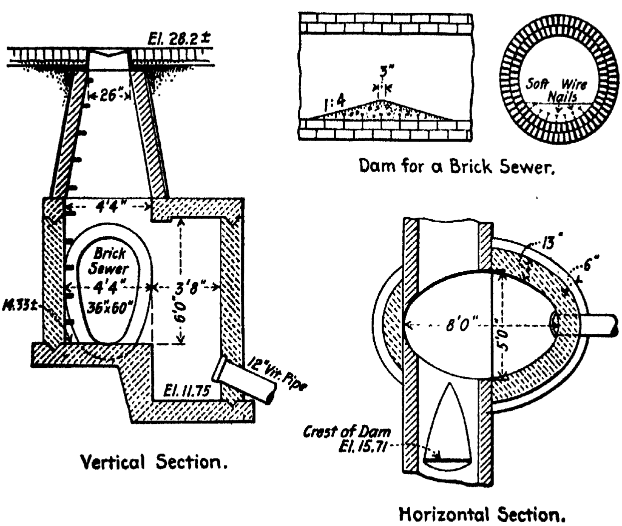


FIG. 277.—Intercepting chamber used at Syracuse.

Material stranded behind the dam is washed out during flood discharges. There have been a number of stoppages in these intercepters due to deposits in front of the small outlet pipes leading from the chamber."

A marginal conduit has been built along the Boston shore of the

Charles River to carry off the storm water from the area tributary to that river. This was necessary because of the construction of a dam across the river between Boston and Cambridge, converting a portion of the shore on either side into unusually attractive property facing a fresh-water basin. The marginal conduit was designed to carry off the first storm wash, which contains most of the dirt from the streets and might pollute the water of the basin in an undesirable way. As the district is closely built up, the area in question is practically impervious and after the first storm-flow had carried off the dirt, it was thought that there would be relatively little more to be expected during the storm. The main conduit was provided, therefore, with a number of overflow chambers, Fig. 278, discharging the excess storm water into short outfall sewers leading to submerged outlets.

These overflow chambers were designed by E. C. Sherman under the direction of Hiram A. Miller. A curtain wall partially separates the overflow chamber from the conduit, so that sewage is drawn from the

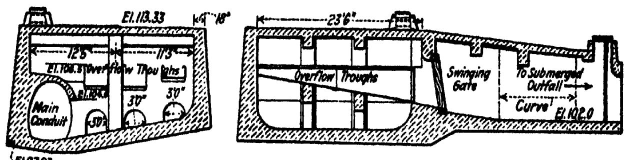


Fig. 278.—Overflow chamber, Boston marginal conduit.

middle part of the stream and floating débris cannot be carried out into the fresh-water basin. The water in the basin is retained at El. 108 and as it was assumed that a loss of head of 0.5 ft. would be caused by the swinging check gate, the crest of the overflow troughs were placed at El. 108.5. The top of the conduit being at El. 106.2, the conduit is under a slight head at times when the overflow takes place. As soon as the troughs are filled, the head on the check gates causes them to swing open and permit flow into the basin to take place through the submerged outlets.

An overflow chamber of unusual arrangement was constructed in Boston about 1899 at a point where a brick sewer was crossed by a large brick conduit at a somewhat lower level, built in that year in order to carry the storm water from an area of about 650 acres, including a small brook known as Tenean Creek. This conduit was 9 ft. high and 10½ ft. wide at the crossing in question. The brick sewer was 3½ ft. high and 2¾ ft. wide; where it crossed the conduit a reducer was constructed and a 36-in. pipe inserted in the arch of the conduit as shown in Fig. 279. The overflow channel starts from a chamber

which is separated from the sewer by a dam and weir; at the outlet of this chamber there is a 3-ft. tide gate to prevent water in the drain

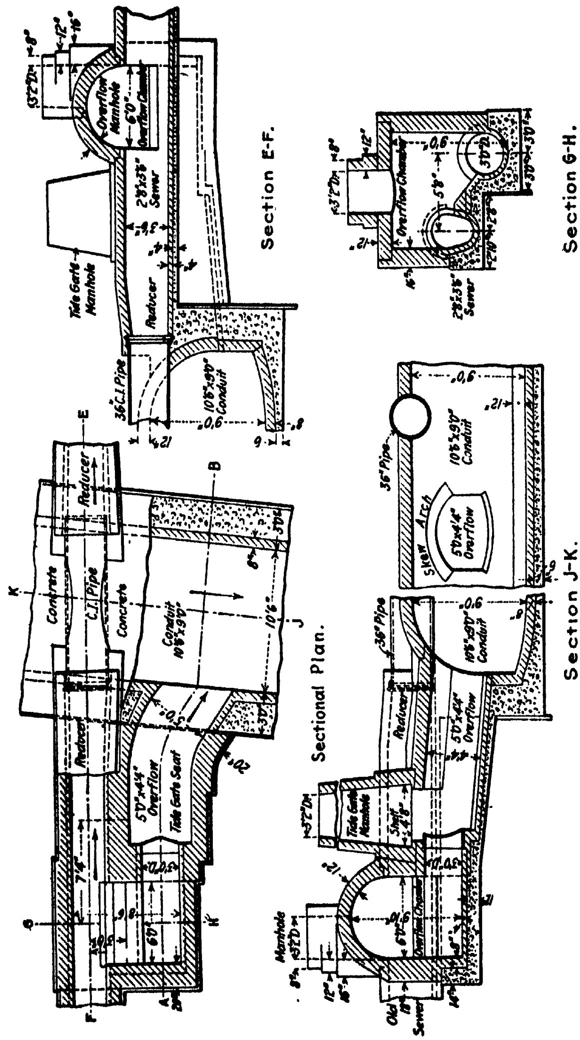


FIG. 279.—Overflow chamber and sewer crossing, Boston.

from passing up into the sewer. Below this gate the overflow is 5 ft. wide by 4 ft. 4 in. high and enters the drain at an angle of 60 deg.

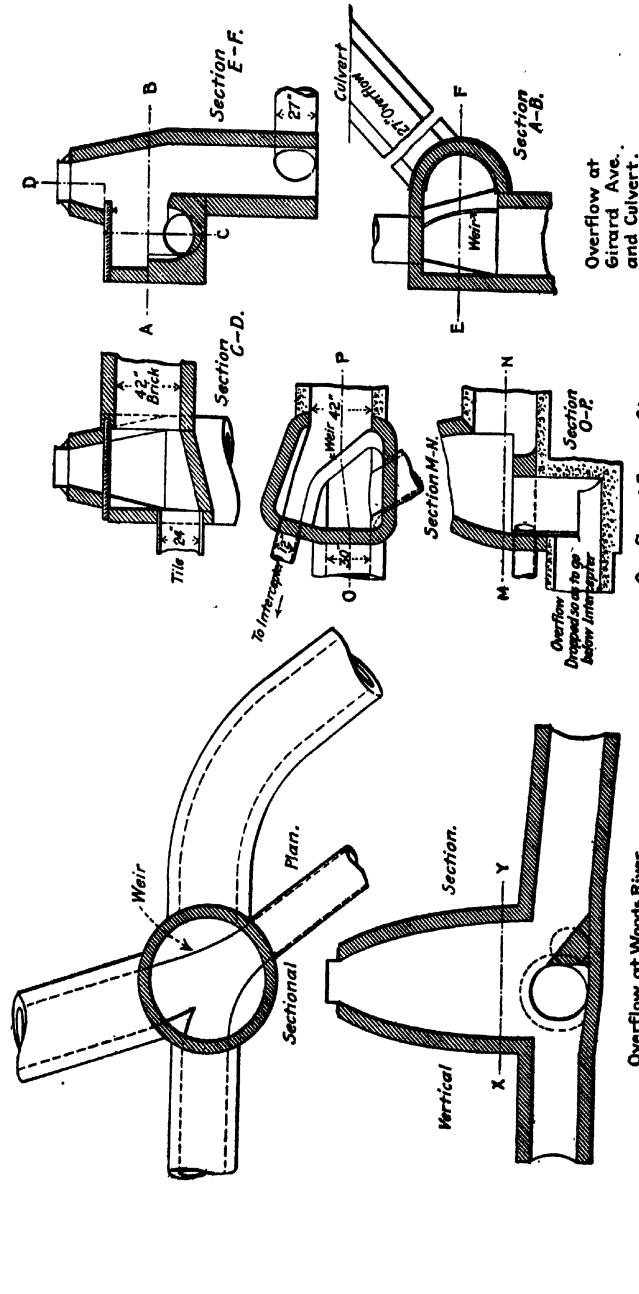


Fig. 280.—Overflow chambers used at Hartford, Conn.

At Hartford, Conn., when a sewer system is being constructed for a district, "a number of local sewers are brought together into a trunk sewer," according to information furnished by Roscoe N. Clark, City Engineer, "which is carried to a point near the intercepting sewer, from which one pipe, to carry the sewage flow, is built to the intercepter, and another, large enough for the storm water, is built to the river, brook or storm-water culvert, as the case may be. In this case a weir is built across the overflow channel with its crest at the top of the

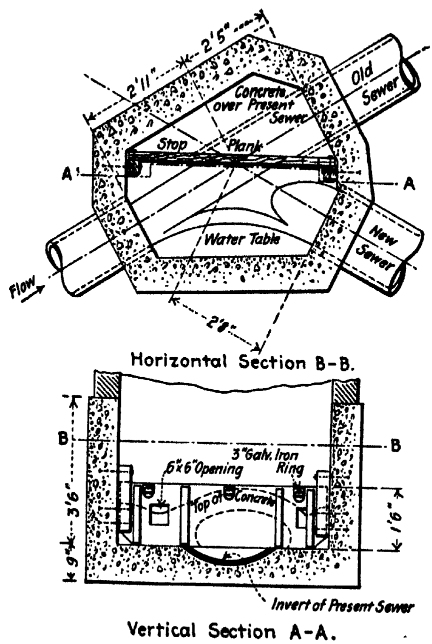


FIG. 281.—Stop-plank regulator, Hartford.

sewage pipe, or above it, if it is desired to have the sewage pipe work under a head, as is sometimes done." Examples of this are shown in the group of storm overflows illustrated in Fig. 280. The overflow at Bonner Street is a rather unusual one, because the overflow has been dropped to go below the intercepter; in most cases the intercepting sewers are the lowest at crossings of this kind.

Where relief sewers must be built to take part of the sewage flowing in old sewers past certain points, use is made of weirs, as in the case of intercepting sewers. For example, in the case of the old Hartford

sewer shown in Fig. 281, it was desired to remove practically all of the storm water but to keep the sewage in the old line. The latter was closed, except for about 3 in. next the invert, by an adjustable stop-plank which was expected to divert everything but the sewage into the

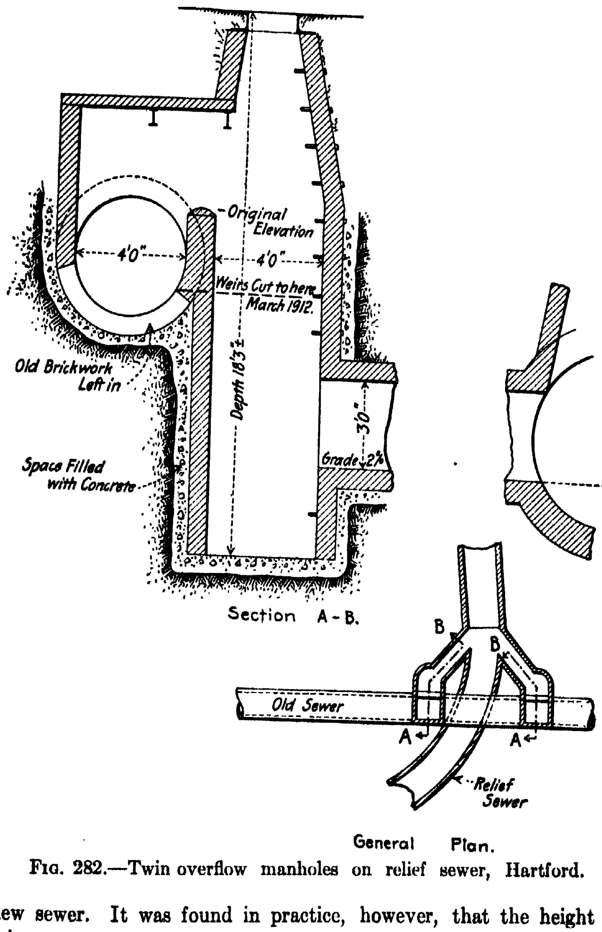


FIG. 282.—Twin overflow manholes on relief sewer, Hartford.

new sewer. It was found in practice, however, that the height of 3 in. was not enough, and 6 in. would have been better to prevent the opening becoming clogged. Another unusual Hartford connection between an old sewer and a relief sewer is shown in Fig. 282. There are two overflow manholes, and the crest of the weir in each, constructed

about 1903, nearly at the top of the old sewer, was cut down after 10 years of service so as to lie only 1 ft. above the invert.

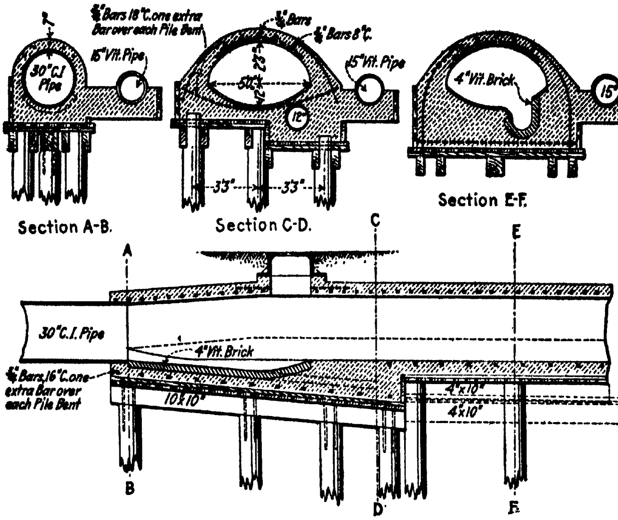


FIG. 283.—Overflow and transformer chamber, Borough of Richmond.

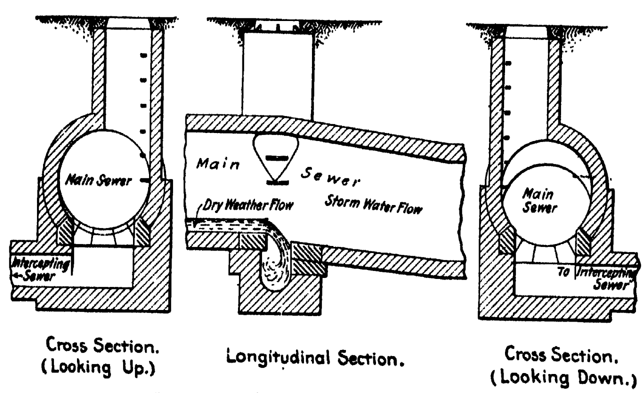


FIG. 284.—Leaping weir at Milwaukee.

An overflow chamber is used for two purposes at the end of a 30-in. cast-iron sewer at Tompkinsville, Staten Island, N. Y. The sewer is likely to operate under pressure at times, and consequently the

sewage must have its velocity checked before it is discharged at the bulkhead line of a pier 455 ft. long. To accomplish this a combined transformer and overflow chamber was built (*Eng. Record*, Feb. 15, 1908). The transformer chamber, Fig. 283, is about 6 ft. long and is at the head of the overflow chamber, so-called, which is really what British engineers call a stilling chamber. It is 65 ft. long and its purpose is to reduce the velocity of the storm-water discharge by providing a greatly enlarged channel. This chamber and the 15-in. storm-water drain serving an adjoining railroad yard end at the bulkhead line, but the dry-weather flow is discharged through a 12-in. cast-iron pipe carried under the pier floor on slings to its outer end.

Leaping weirs consist of openings in the inverts of sewers so constructed that the ordinary flow of sewage proper falls through the open-

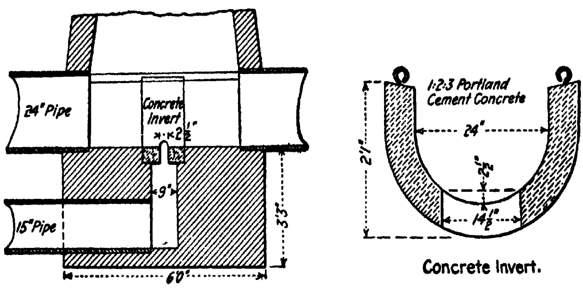


FIG. 285.—Leaping weir for pipe sewers, Cleveland.

ings and passes to the intercepters. At times of storm, the increased velocity of flow causes most of the sewage to leap the openings and pass on down the sewers to the storm outlets. The first use of the device is commonly attributed to J. F. Bateman, designer of the first water works of Manchester, England.

The first use of the leaping weir in this country is believed to have been in Milwaukee, where 12 branches to the Menomonee intercepter were connected by means of leaping weirs in 1887 and subsequent years. One of these connections is shown in Fig. 284.

The most simple type of leaping weir is that in which the dry weather flow drops through a slit cut across the invert of the combined sewer. Such a weir used in Cleveland, Ohio, is shown in Fig. 285. The type is used on the smaller sewers and is known locally as the weeping weir; for larger sewers the manhole shown in Fig. 276 is preferred. Regarding the former Mr. Estep states: "In the smaller systems this type is about as satisfactory as can be installed. We make calculations as to the amount of dry-weather flow in each case from the acreage, and

TABLE OF DIMENSIONS

Size	A	B	C	Ra	Rb	Style
15 in.	13½"	5½"	10¾"	7½"	7"	A
18 in.	16½"	7"	12"	9"	8½"	B
24 in.	22 9/16"	9½"	14⅜"	12"	11½"	C

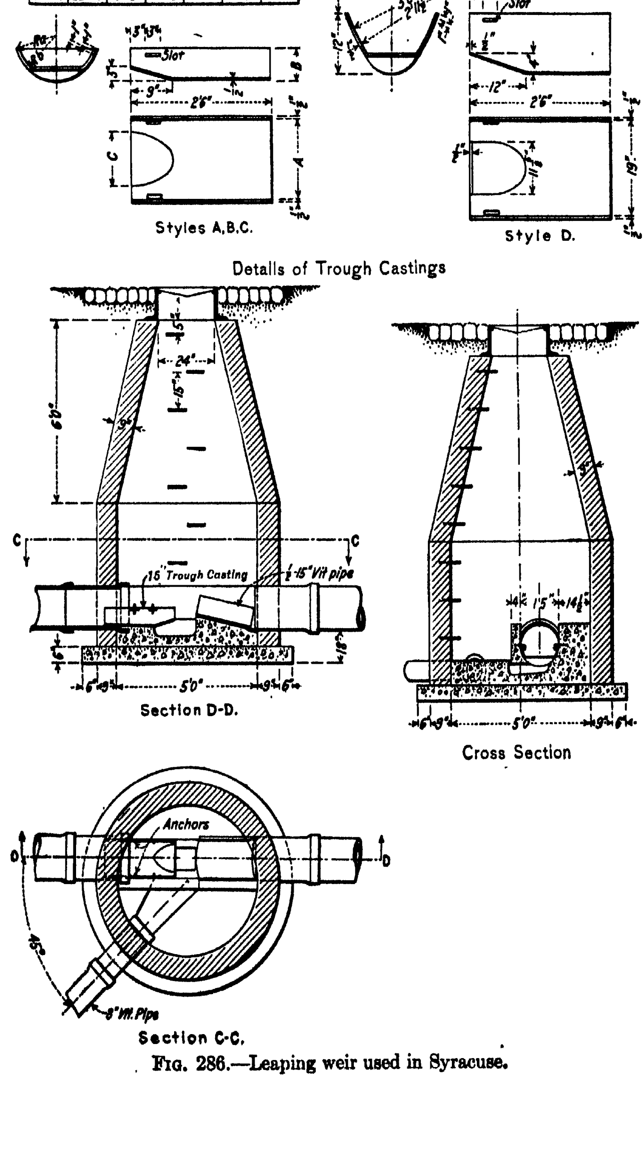
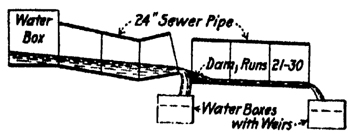
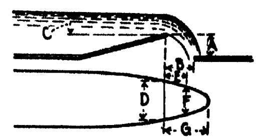


FIG. 286.—Leaping weir used in Syracuse.

then compute the size of the opening required to pass this amount of sewage."

In one branch of the intercepting sewer system of Syracuse, N. Y., now (1913) under construction, leaping weirs are used at the connections of existing sewers with the intercepters and float regulators are also employed to safeguard the intercepters against surcharge. The type of weir employed at Syracuse is shown in Fig. 286. It is formed of a channel pipe, inclined upward so as to contract the flowing stream and give a spouting effect. During dry weather when the quantity of sewage is small and the velocity slight, the sewage merely drops over the weir into the channel leading to the intercepter. At times of storm flow, the





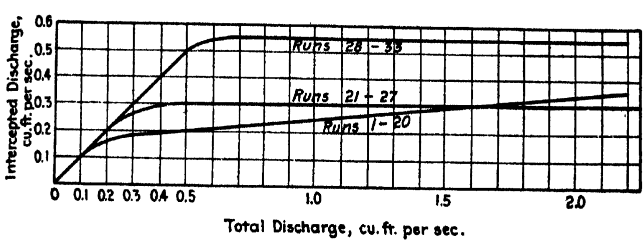


FIG. 287.—Syracuse leaping weir experiments.

increased velocity causes the greater part of the sewage to leap the opening, and it is caught in a cast-iron trough which may be adjusted in position, so as to vary the width of opening. Several of these weirs have been in use for periods up to 3 years, and have given good satisfaction. The only difficulty experienced has been when foreign matters, such as rags and sticks, have clogged the opening.

Tests were made upon a model weir of this type under the direction of Glenn D. Holmes, Chief Eng. of the Syracuse Intercepting Sewer Board, the results of which are given in Table 166 and Fig. 287.

Another method of constructing a leaping weir with an adjustable width of opening, as suggested in Moore and Silcock's "Sanitary Engineering," is shown in Fig. 288. The following analytical treatment of the device is taken from that source, where it is credited to Prof. W. C. Unwin:

TABLE 166.—SYRACUSE EXPERIMENTS WITH LEAPING WEIRS, FIG. 287
(*Glenn D. Holmes*)

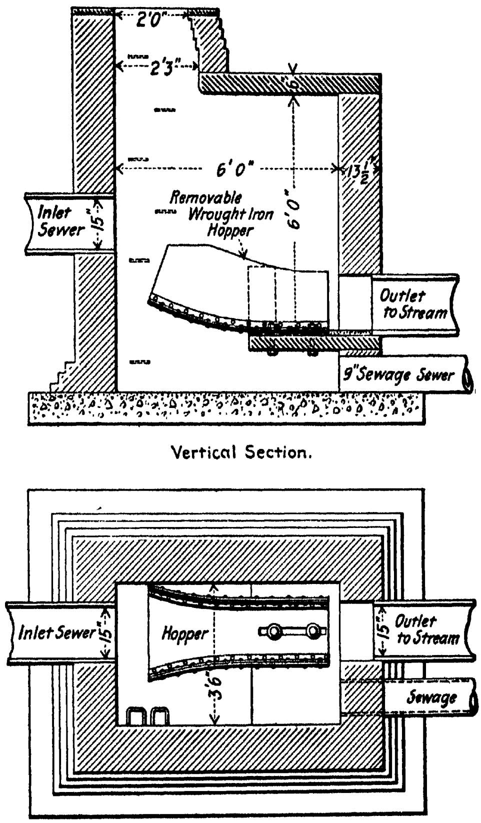
Run	Metered supply, sec.-ft.	Intercepted water, sec.-ft.	Dis. at outlet, sec.-ft.	Dimensions, ft., Fig. 287.						
				A	*B*	*C*	*D*	*E*	*F*	*G*
1	0.5	0.2	0.3	.....	.....	.....	.....	.....	.....	.....
2	0.15	0.15	0.0	0.4	0.4	0.17	1.08	0.21	0.75	0.4
3	0.26	0.2	0.07	0.4	0.4	0.20	1.21	0.25	1.17	0.4
4	0.36	0.21	0.21	0.4	0.4	0.24	1.27	0.25	1.00	0.4
5	0.70	0.22	0.48	0.4	0.4	.....	.....	.....	.....	.....
6	0.124	0.12	0.0	0.4	0.4	0.19	1.04	0.25	0.8	0.4
7	0.512	0.2	0.29(?)	0.4	0.4	0.29	1.35	0.25	1.2	0.4
8	0.438	0.18	0.12	0.4	0.4	0.25	1.25	0.25	1.05	0.4
9	0.407	0.19	0.19	0.4	0.4	0.27	1.3	0.25	1.12	0.4
10	0.708	0.19	0.5	0.4	0.4	0.34	1.5	0.25	1.3	0.4
11	0.862	0.25	0.62(?)	0.4	0.4	0.36	1.50	0.25	1.4	0.4
12	0.4	0.16	0.06	0.4	0.4	0.20	1.17	0.25	0.93	0.4
13	0.268	0.18	0.06	0.4	0.4	0.21	1.2	0.25	0.96	0.4
14	0.443	0.21	0.22(?)	0.4	0.4	0.27	1.3	0.25	1.10	0.4
15	0.731	0.22	0.40	0.4	0.4	0.31	1.4	0.25	1.25	0.4
16	0.758	0.22	0.52(?)	0.4	0.4	0.34	1.5	0.25	1.4	0.4
17	....	....	....	0.4	0.4	.....	.....	.....	.....	0.4
18	....	0.28	0.67	0.4	0.4	0.39	1.55	0.25	1.4	0.4
19	....	0.28	1.09	0.4	0.4	0.44	1.7	0.25	1.5	0.4
20	....	0.32	1.13	0.4	0.4	0.44	1.7	0.25	1.55	0.4
21	0.2′ dam	0.31	0.67	.....	.....	.....	.....	.....	.....	.....
22	0.2′ dam	0.31	1.05	.....	.....	.....	.....	.....	.....	.....
23	0.2′ dam	0.27	0.05	.....	.....	.....	.....	.....	.....	.....
24	0.2′ dam	0.29	0.11	.....	.....	.....	.....	.....	.....	.....
25	0.2′ dam	0.31	0.29	.....	.....	.....	.....	.....	.....	.....
26	0.2′ dam	0.31	0.93	.....	.....	.....	.....	.....	.....	.....
27	0.2′ dam	0.20	0.00	.....	.....	.....	.....	.....	.....	.....
28[1]	0.33	0.33	0.00	.....	.....	.....	.....	.....	.....	.....
29[1]	0.55	0.52	0.01	0.3	0.6	0.27	1.4	0.25	1.25	0.4
30[1]	....	0.52	0.05	0.3	0.6	0.30	1.45	0.25	1.2	0.4
31	....	0.55	0.20	.....	.....	.....	.....	.....	.....	.....
32	....	0.55	0.28	.....	.....	.....	.....	.....	.....	.....
33	....	0.55	0.69	.....	.....	.....	.....	.....	.....	.....

[1] Dam 0.2 ft. high in discharge pipe.

Let h be the head of water over the upper lip of the opening, x the horizontal distance from the upper lip to the edge of the lower lip on the farther side of the opening, and y the vertical drop from the upper lip to the edge of the lower lip, and t the time for a particle of water to pass from one lip to the other. For practical purposes, the mean velocity of the water will be

$$v = 0.67\sqrt{(2gh)}$$
$$y = 0.5gt^2$$
$$x = 0.67t\sqrt{(2gh)}$$
$$y = 0.56x^2 \div h$$

This gives the width for any given difference of level which the jet will just pass over with a head h. If in addition there is a velocity of approach, h must include the head necessary to give that velocity viz., $v^2/2g$.



Sectional Plan.

FIG. 288.—An adjustable leaping weir.

Silt Chambers.—One of the objections to practically all diverting devices is the fact that silt is diverted into the intercepting sewers and is also likely to accumulate in the space reaching from the weir to the intercepting sewer. Not only the silt brought down by dry-weather flow, but that carried or rolled along the bottom by storm water is accumulated

in this space. This is likely to give rise to deposits. Even in so carefully designed an overflow as that at Cleveland, where there is no dead space behind the weir, the silt dragged along the bottom by storm water cannot pass the weir but must be carried on into the intercepting sewer.

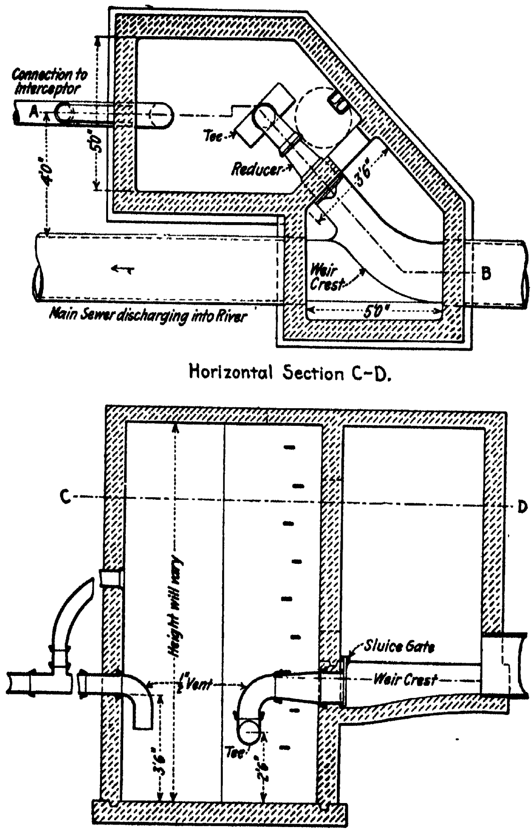


FIG. 289.—Overflow and silt basin, Harrisburg.

In some cases particular care has been given to the design of basins in which the silt carried down with the sewage can be retained and prevented from passing into the intercepting sewer. Two different ideas have been followed in designing such basins.

In one case a sump is constructed to retain the silt, forming practically a catch-basin from which the silt can be removed from time to time. An example is shown in Fig. 289, an illustration of an overflow and silt basin used at Harrisburg, Pa., built from the designs of James H. Fuertes, consulting engineer, New York City. The drawing requires no explanation.

In the other type a depression is formed in the sewer, above the regulator, so shaped that the silt will be scoured out by storm flow and carried down the storm sewer to the overflow. One of the best examples of this type is seen in the illustrations of the storm overflows at Washington, D.C., Figs. 270 and 271. In the first of these overflows the silt chamber consists of a depression in the invert of the main sewer. This is sufficient to retain the silt brought down during ordinary times. At times of storm, when the regulator gate is closed, the high velocity scours out the accumulated silt and carries it over the dam to the storm water outlet. In the second illustration there is a silt basin of considerable size in the chamber above the regulator gates. By opening a sluice gate at the side of the chamber at times of storm flow, the silt can be forced into the storm sewer itself.

An objection to either of these designs is that an opportunity is afforded for organic matter to accumulate during low flows and to putrefy, thus forming offensive pools of sewage.

OUTLETS

Strictly speaking, the outlet of a sewerage system is the end of an outfall sewer at which the sewage is discharged. There may be a number of these outlets in case the city has several storm water outfalls or overflows. In every case, the object should be to discharge the sewage at a point where its presence will cause no offense; the disposal of the storm water is not so difficult because it contains less organic matter and is not delivered continuously. Where the water is quiet the outlet of the outfall sewer is usually submerged to a considerable depth, while if the sewage is discharged into a stream flowing rapidly at all times, the outlet need not necessarily be submerged, provided the sewage passes into the stream at a point where it is certain to be carried away and dispersed rapidly. In the case of outlets in tidal waters, the fact that it is generally impracticable to place them so high that they will not be entirely sealed at high tide, results automatically in checking the discharge of sewage during the portion of the tidal flow when it is likely to be swept back along the shore, and accelerates the discharge when the tide is going out and the hydraulic grade of the outfall is, therefore, being steadily increased.

A different outlet is sometimes built for combined sewers than for

those carrying nothing but sewage, because the latter must be discharged with much greater precautions to prevent nuisance than the storm water flowing from combined sewers or drains. A combination of these conditions is illustrated in Fig. 290, from *Engineering Record*, April 8, 1911. This outlet was built at Minneapolis, where the level of the Mississippi, into which the sewage is discharged, fluctuates materially. The conditions made it practicable to build a double outlet, by which the dry-weather flow is carried out farther into the stream and to a lower level than the storm water. Two 15-in. cast-iron pipes run out below the paved apron in front of the storm-water outlet, and discharge the dry-weather sewage 5 ft. below low-water level in the river. The invert of the storm water sewer is 9 in. below the high-water level in the river, so that the sewer will have a free discharge at all times.

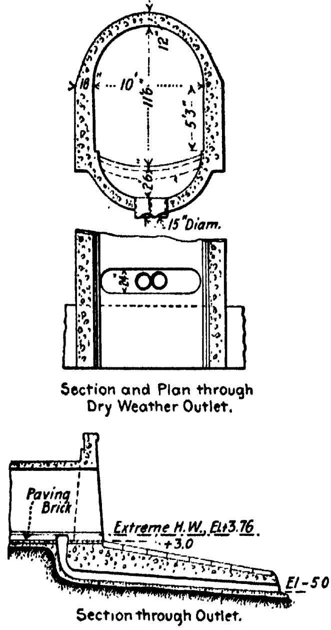


FIG. 290.—Dry-weather outlet, Minneapolis.

Much the same plan is followed in the outlets of the sewerage system of Winnipeg, built from the plans of Col. H. M. Ruttan. The outfall sewers are built of concrete until they approach the banks of the rivers into which they discharge. Each outfall is then continued by a wooden sewer running out on pile bents at an elevation of 3 or 4 ft. above the river. Its outer end is closed by a large flap door, which floats upward when the river is in flood. About 10 ft. from the outlet end, a small pipe drops from the invert and is then carried forward on piles 50 ft. or more beyond the end of the main outlet, to take the dry-weather sewage well out into the stream in times of low-water. These outlets are protected against the heavy ice flows by a sloping ice-break of 6 × 6-in. timber, laid so as to carry the ice over the structures.

Where the sewage must be carried out into comparatively deep water, the outfall sewer is generally a cast iron or steel pipe ending in a quarter bend or a tee, by which the sewage is discharged upward. A typical

outlet of this character was built in 1913 to carry the effluent from the Rochester sewage treatment works into Lake Ontario. The pipe is 66 in. in diameter and made of half-inch plate, the straight portions being of the Lock-bar type with single riveted seams every 30 ft., and the bends of short sections with double riveted longitudinal seams. The submerged portion of the pipe was laid in a dredged trench 8 ft. deep until a depth of 35 ft. was reached, when the trench was shallower. The minimum back-fill over the pipe was 2½ ft. The pipe terminates in a timber crib 7000 ft. from the shore, and the discharge is at a point where the water is about 50 ft. deep. The crib or outlet structure is 46 ft. square by 24 ft. high, built of 12 × 12-in. hemlock timbers laid to form 25 pockets,

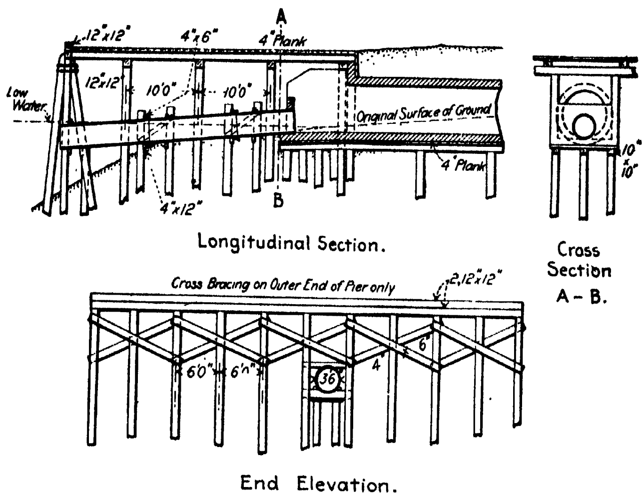


FIG. 291.—Outlet of joint trunk sewer, New Jersey.

which are filled with stone except where they are occupied by the pipe. The bottom of the crib is 3 ft. below the bottom of the lake and is surrounded with riprap extending 10 ft. up the sides of the structure. The top is 26 ft. below the mean low-water surface of the lake. The pipe discharges 10 ft. above the bottom of the lake, being raised as it passes through the crib. Built into the crib near the outlet, is a three-way Tee, the side openings being 38 in. in diameter. This was placed to provide for future extensions in case it is deemed necessary to discharge the effluent at more than one point, to promote more thorough dilution.

The outlet of the joint trunk sewer of Northeastern New Jersey, on the shore of Staten Island Sound, is illustrated in Fig. 291. Like many

of the outlets in the vicinity of New York Bay, it is below a wharf, which was constructed in this case in return for permission to establish the outlet at this place. The wharf is 60 ft. wide and about 40 ft. long, its base at the dock line making an angle of about 75 deg. with the axis of the sewer. The 72-in. brick sewer terminates in a brick chamber 7-1/2 ft. square at the upper end of the wharf, from which a 36-in. cast-iron pipe extends a distance of 36 ft. to the bulkhead of the wharf, where its crown is 2-1/2 ft. below low-tide elevation. This pipe is carried on piles independently of the wharf and is said to have enough capacity to discharge at the dock line, where there is a strong current, a volume of sewage equal to that delivered to the chamber by the 72-in. sewer. Owing to some doubt as to the stability of the foundations in this vicinity, the last 80 ft. of the brick sewer rests on a 4-in. plank floor 8 ft. wide, supported on three 10 × 10-in. stringers which are carried by three rows of piles. This structure was designed by Alexander Potter, Chief Eng. of the commission representing the seven communities interested jointly in the work.

The outlet of the southern outfall system of Louisville is shown in Fig. 292. It is at the end of a sewer 10 ft. 1-1/2 in. high and 10 ft. 7½ in. wide. It includes a drop chamber 93 ft. long, built on concrete piles on the steep incline running down to an outlet structure 56 ft. long, the foundations of which rest on rock.

The crown of the outlet will be below the surface of the water in the river at all times after the proposed 9-ft. stage of the Ohio recommended by the War Department has been established by Congress. Before that time there may be occasions when the outlet will be partially exposed during extreme low water; during floods the river rises many feet above the outlet, the maximum being probably about 70 ft.

In determining the size of the drop and outlet structures, a hydraulic grade was assumed from the top of the sewer at the upper end of the drop chamber to the surface of the water in the river when at El. 415, or 32 ft. above the elevation for the 9-ft. river stage. This elevation is rarely exceeded during freshets in the winter; in June the height of the water has exceeded this stage only twice in 35 years, and remained above it for only a very short period of time then. Storms of great intensity are not frequent in this locality except in June, July and August, and are very rare during the winter. The possibility of the occurrence of rainfalls of such high intensity as to tax the capacity of the sewer, occurring at a time when the river is above El. 415, was considered by the engineers in charge of the work to be very remote, and for this reason it was believed to be safe to base the design of the drop and outlet structures upon the hydraulic grade mentioned. The outlet structure will generally be submerged in the river, and occasionally at times of extreme floods the entire drop chamber and even the outfall sewer itself will be submerged for some distance. It was considered impossible to

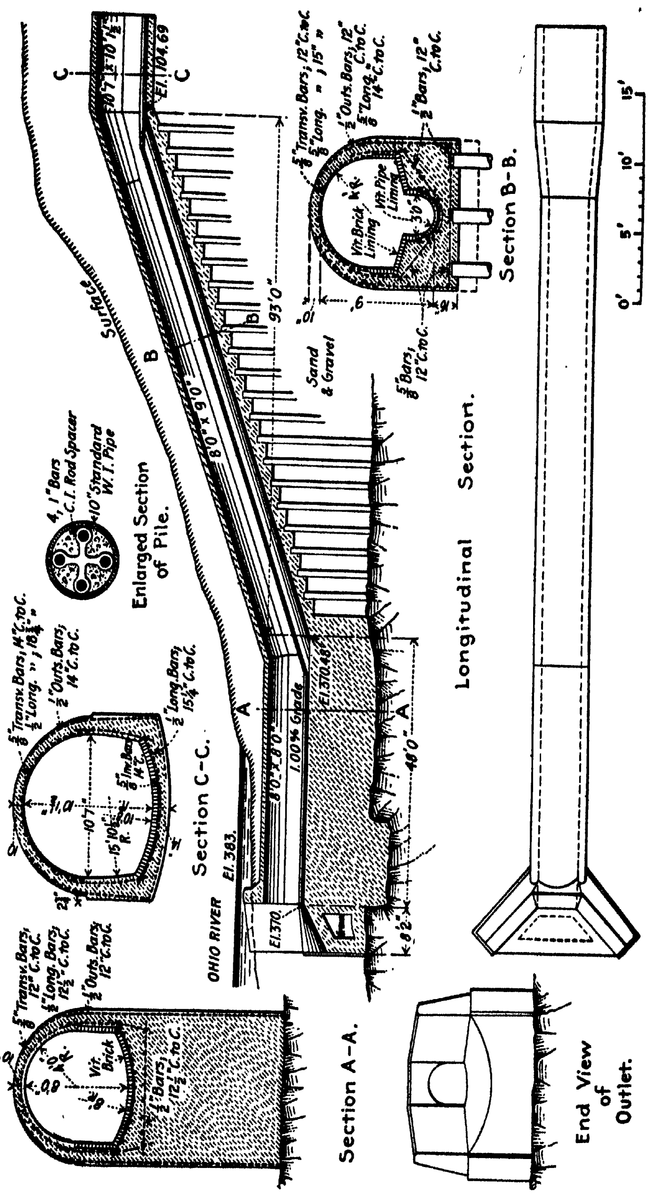


FIG. 292.—Southern outlet structure, Louisville.

provide adequate drainage in the city during storms of great severity occurring at a time when the river is at an extreme flood stage. Such conditions are so rare that they must be construed as an "act of Providence," for which the city should not be expected to make provision.

There have been indications of a strong tendency of the river bank to move toward the river after the falling of the water in the late spring or summer. The bank is composed, to a large degree, of silt, which becomes saturated during high stages of the river, and is very heavy when wet, possessing little stability. Underlying the silt is a bed of coarse sand and gravel, through which large quantities of water are flowing continually toward the river. The action of this water at the surface of the gravel probably tends to assist the sliding action of the silt above. In anticipation of any such action and its consequent effect upon the sewer at its outlet, the foundation was carried down to bed rock, as illustrated. For a short distance, 15 ft., the rock was excavated to a depth of 4 or 5 ft. and the foundation carried down in this pit to form a key to guard further against any movement.

The drop chamber was built on piles to assist in resisting any possible movement, as well as to support the structure in case, by any chance, it should be undermined by the action of the river. These piles extend to the rock where it is within 20 ft. below the masonry, and 20 ft. into the ground further up the bank, in all cases penetrating a long distance into the gravel underlying the silt.

The outlet structure is 8 ft. wide and 8 ft. high, with a semi-circular arch, vertical side walls nearly 3 ft. high, and a comparatively flat but curved invert. At its outer end two wing walls were built out into the river, each making an angle of 45 deg. with the axis of the sewer.

The drop chamber has an arch, short side walls and invert of the same dimensions as those of the outlet structure. In the center of the invert, however, there is a channel 3 ft. wide and 2 ft. 10 in. deep, lined with half-round vitrified sewer pipe. This channel is for the dry-weather flow, which will have a very high velocity. The pipe lining was used rather than vitrified brick, because of the absence of longitudinal joints, at which inverts on steep grades show the greatest amount of erosion, and for its good wearing qualities. On account of the velocity which will be obtained during the lower stages of the river, both the outlet and drop structures have been lined with vitrified brick to the top of the side walls.

The outlet of the northwestern sewer in Louisville is of the same general type but illustrates a different method of supporting such a structure. The cross-section in all places is 6 ft. 8 in. wide, by 6 ft. high, Fig. 293. The outlet is submerged by the proposed 9 ft. stage in the Ohio River. For a distance of 77 ft. from the headwall, the grade is

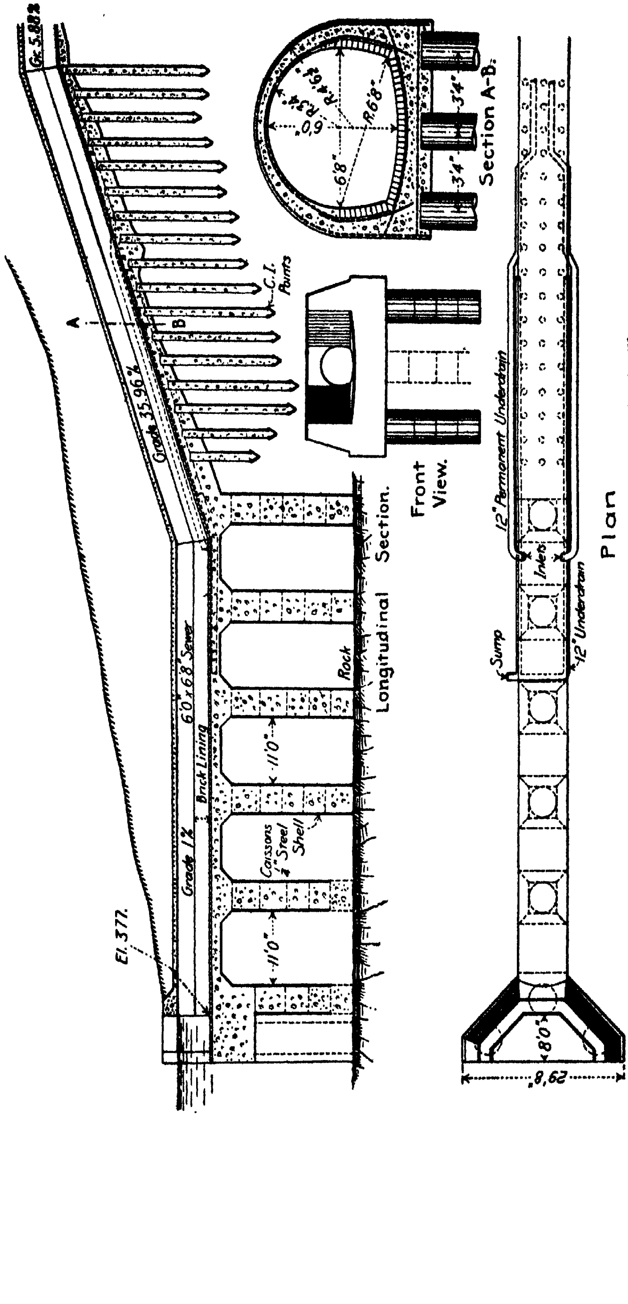


FIG. 293.—Northwestern outlet structure, Louisville.

1 per cent. Then for a distance of 78 ft. the slope is 35.96 per cent., and finally for a distance of 256 ft., the slope is 5.88 per cent.

The mouth of the sewer is protected by head and wing walls of massive concrete, and the lower portion of the sewer rests on concrete columns built within steel caissons. The shell of the caisson was built of quarter-inch plate in 4- or 5-ft. widths, and each caisson was 5 ft. in diameter. These supports varied from 20 to 25 ft. in length. Each of them was surmounted with a cap which left 7 ft. of unsupported sewer between piers. The minimum thickness of the concrete of the invert of the sewer, between the column caps, is 2 ft. The illustration shows the supports as planned; as a matter of fact, these columns proved so much superior, in this work, to the reinforced concrete piles, that the last four bents of the latter were omitted and a column substituted for them.

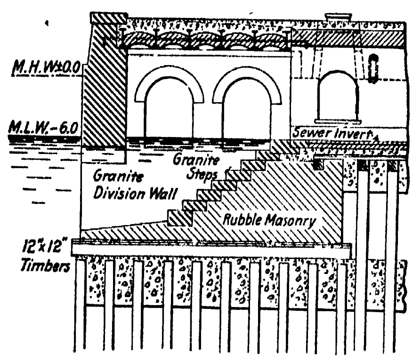


FIG. 294.—Outlet of Broadway outfall sewer, Borough of the Bronx.

The drop chamber is carried on Simplex concrete piles with four 3/4-in. twisted reinforcement bars extending from their top into the concrete of the invert of the chamber.

The outlet of the Broadway outfall sewer, Fig. 294, in the Borough of the Bronx, New York City, is 57 ft. long on its outer side along the river, 41 ft. long on the inner side where the sewer joins it, and 21-1/2 ft. wide along the cross-section shown in the illustration. It has a total height of 22 ft. and is constructed of granite ashlar masonry with a heavy concrete roof. The invert of the end of the outfall sewer is at mean low-tide level. This sewer has a twin semi-circular section. The sewage escapes through four openings in the front wall, each 8 ft. wide and 8 ft. high, and as the arch of the opening in each case is below mean low water, it is expected that no floating matter will leave the outlet chamber. The illustration is from *Eng. Record*, Nov. 11, 1905.

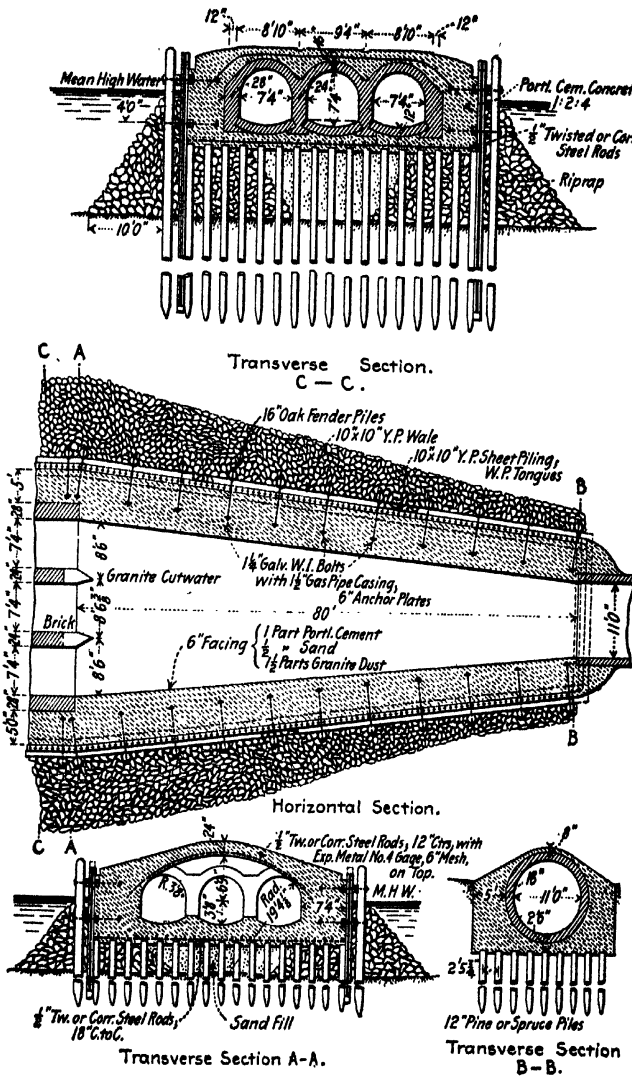


FIG. 295.—Increaser chamber, Brooklyn.

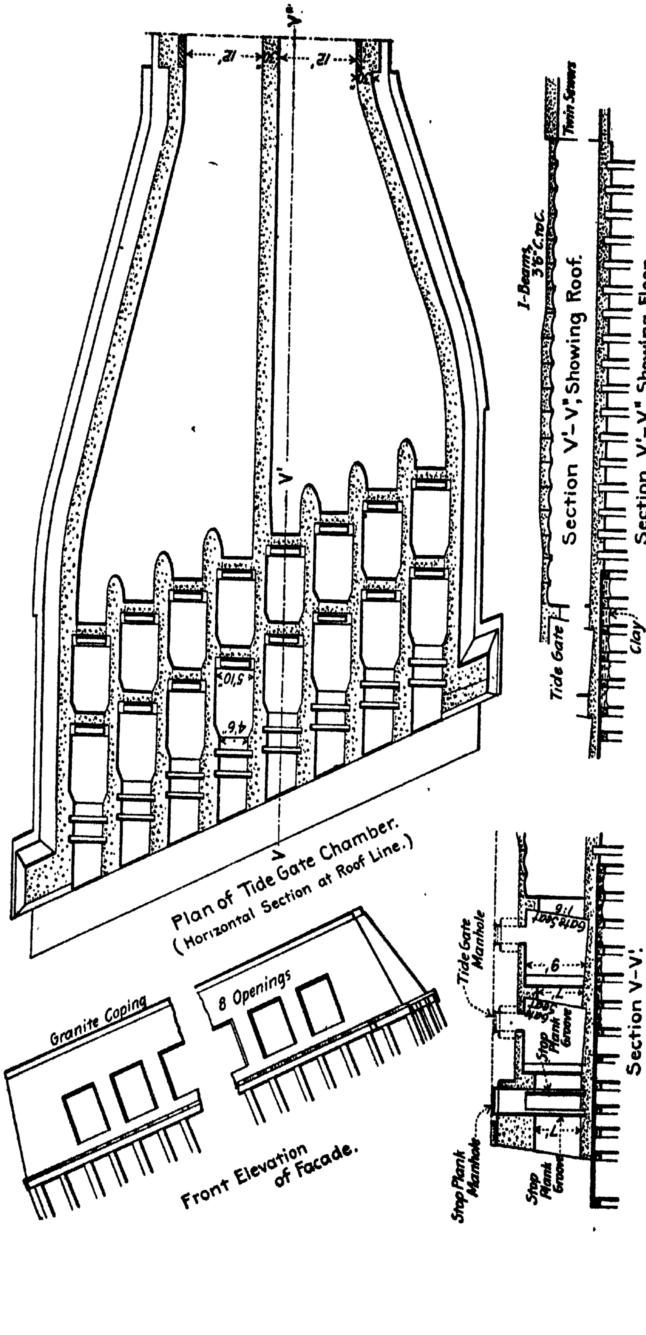


FIG. 296.—Storm-water outlet at Washington.

The outlet of the 92nd Street sewer in the Borough of Brooklyn, New York City, shown in Fig. 295, includes an increaser chamber, 80 ft. long, extending from the end of an 11-ft. sewer where it emerges from a tunnel to a triple sewer having three basket-handle sections carried out on a riprap embankment far enough for the sewage to be discharged into a portion of the Narrows having swift tidal current. The whole structure is very heavy, owing to the strong current to which it is subjected, and also to the fact that it may be utilized in the future by the Municipal Department of Docks and Ferries. The bottom at the site of the outlet is coarse shingly gravel, with a lower stratum of compact sand and gravel.

Fig. 296 is the outlet structure for storm-water of the high-level intercepting sewer in Washington. The water is brought to the structure in two 12-ft. channels with arched masonry roof. The outlet structure is provided with a roof of concrete between I-beams spaced 3 ft. 6 in. apart. The outlet has a longitudinal wall 30 in. wide which supports the inner end of these beams along the center line of the structure. The general arrangement of the structure is shown so well in the illustration, from drawings furnished by Asa E. Phillips, Superintendent of Sewers of the District of Columbia, that no explanation is necessary. The entire structure is carried on piling spaced 3 ft. 6 in. on centers in each direction for the most part.

TIDE GATES

Wherever an outlet ends at a body of water subject to considerable fluctuations in level and it is necessary to prevent this water from entering the sewer, a backwater or tide gate is employed. This consists of a flap hung against a seat which inclines backward as it rises. The hinges may be at the top in case the gate consists of a single leaf, as is usually the case, or they may be at the side, in case the gate consists of two leaves.

One of the earliest types of large tide gates to work satisfactorily was that designed by Otis F. Clapp while in charge of the sewer department of Providence, R. I., of which place he subsequently became city engineer. This is shown in Fig. 297, from *Eng. Record*, Aug. 29, 1896. Ordinarily the entire flow from the 24-in. lateral sewer dropped through the rack, *R*, in the bottom of the chamber, *A*, into the intercepting sewer at a lower level. When the volume of sewage became too great for the intercepting sewer, it rose in the chamber, *A*, and swung open the gate, *G*, so as to obtain an outlet through the storm sewer into Narragansett Bay. The gate, G, revolved about its axis, *B*, and also about the axis, *C*, so that it moved freely even with a slight flow of sewage from the chamber *A*. When the tide backed up to the storm sewer, the gate was pressed firmly against its seat. The adjustment of the gate in position was readily

made by means of the nuts *D*. The unusual feature of this design is the use of very long links for holding the gate.

For a number of years, the larger tide gates in Boston were frequently hinged at the sides. Each gate consisted of two leaves, and as the seat was inclined with the top inward, as usual, when the gate opened it rose slightly as well as moved outward. Consequently it tended to fall back again when the pressure of the outflowing storm water and sewage decreased. In order to make certain that this closing should take place, it was customary to hitch to the back of each leaf a "bridle chain"

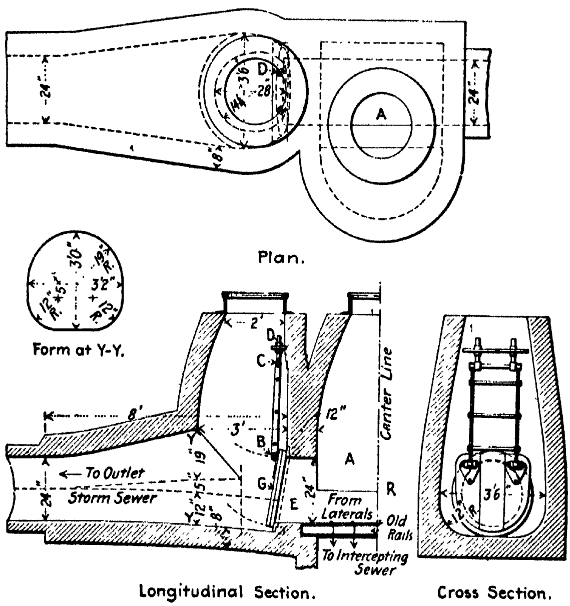


FIG. 297.—Tide gates at Providence.

hanging loosely from a substantial eye in the roof of the gatechamber. On the lowest part of the chain as it hung between the roof and the closed gate was a heavy weight. This bridle chain tended to close the gates when they were open. This type of gate was known as the "barn door" and has now been abandoned on account of the number of adjustments which were necessary to keep it in condition and because the bridle chains accumulated large masses of floating substances which interfered with their proper operation.

The type of tide gate and chamber now used in Boston is shown in

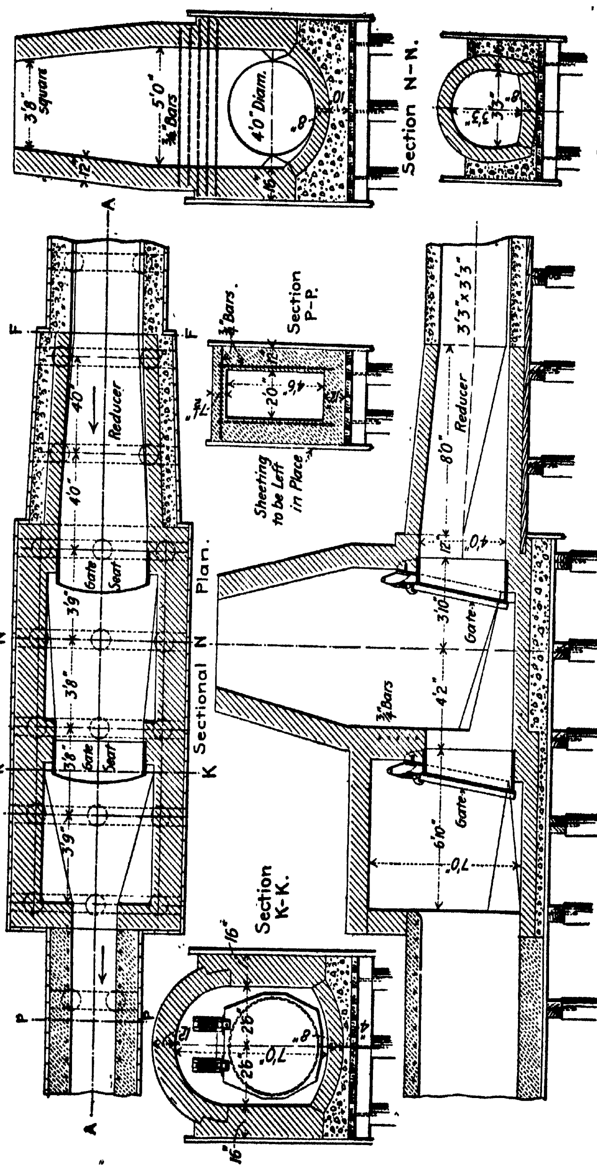


FIG. 298.—Boston tide gates, 1911 type (patented).

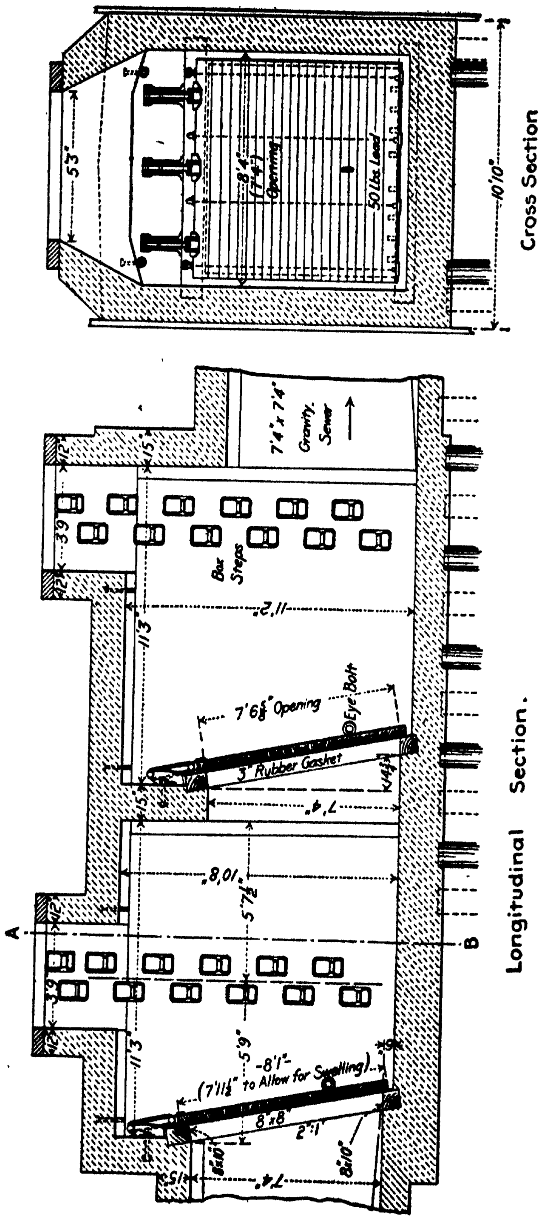


Fig. 299.—Tide-gate chamber, Boston, with Dodd gates (patented).

Fig. 298. It will be observed that the wooden gate rests directly on the end of the cast-iron seat. Formerly the seat was a heavy wooden frame with which the flap made a tight joint by means of a rubber gasket slightly recessed along each edge, so that the nails used in hold-

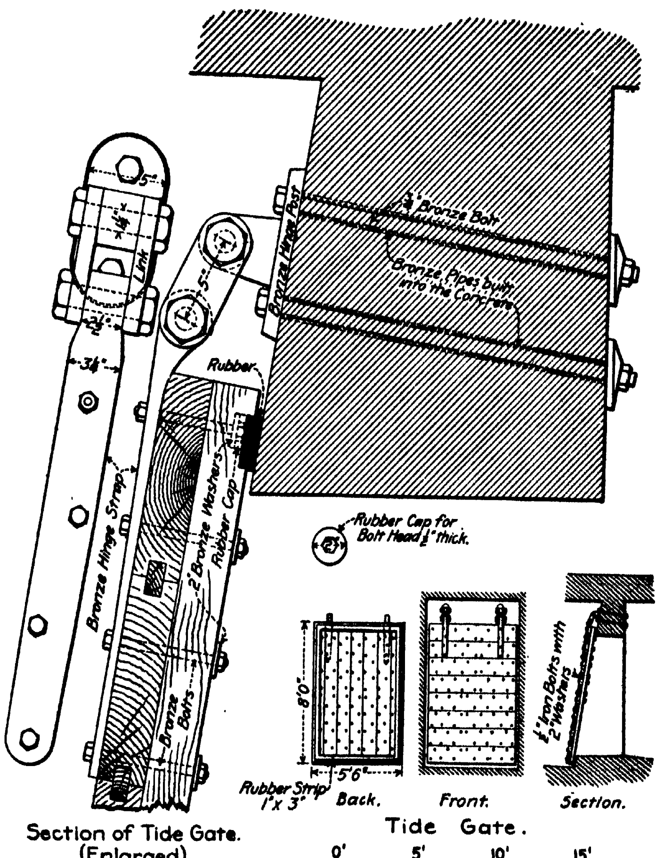


FIG. 300.—Details of Washington tide gates.

ing it to the wood would not project and interfere with the proper compression of the rubber when the gate was subject to back pressure. The type of gate illustrated is regularly made for 12, 18, 24, 36, 48 and 60-in. outfalls, by the Gibby Foundry Co., from the designs of C. H. Dodd

who patented the novel features. The timbers are held together by vertical binding rods, and in the lower part of the flap a number of displacement weights are inserted, which serve the same purpose as the bridle chain in the older form of side-hinged gates. A larger gate of the same general type, also designed by Dodd for use in Boston, is shown in Fig. 299.

The tide gate used in Washington in the structure illustrated in Fig. 296, is shown in Fig. 300. Asa E. Phillips, superintendent of the Sewer Department of the District of Columbia, informed the authors that these gates are made of double cross lapped 3 × 12-in. Georgia pine, shipped directly from the southern mill where it is cut and kept under a damp cover until ready to place. The contact is made on the concrete gate seat by a rubber strip 3 in. wide and 1 in. thick, set half into the wood. These gates have been very effective, requiring scarcely any attention, and have always been substantially water-tight. They required no renewal or repair for 5 years after their erection, and very little for a number of years longer.

The operation of tide gates by hand has been attempted at times, as at Hoboken, N. J., where there were three thus served in 1912 and one which was a simple automatic flap gate like those in Boston. James H. Fuertes found in that year that there was an attendant at each manually operated gate all the time, 12-hour shifts being in force, and each man followed a system of his own in managing the gates. At one place the gates are opened an hour after high tide and closed an hour after low tide, with some variation during very high or low tides. At another place the gates are opened from 3-1/2 to 4 hours after high tide and closed from 2 to 2-1/2 hours after low tide. Observation showed Mr. Fuertes that the proper time to open the gates was directly after high tide and for closing them 2 hours after low tide, and that the automatic gates would probably give better service than manual operation of the kind likely to be provided.

VENTILATION

For many years the provision of special structures to aid the ventilation of sewers was one of the most troublesome tasks of the designer. The gravity of the problem is probably not appreciated to-day, when the necessity of good grades and construction is so generally recognized that the conditions which frequently faced a city engineer not much more than 25 years ago are hardly to be believed. When the sewerage systems frequently contained old sewers which had either been so constructed as to cause the formation of banks of sludge and pools of septic sewage, or had been allowed to fall into such a dilapidated condition that the same evil results followed, it is not suprising that engineers

as well as the general public had good reason for believing that there was such a thing as "sewer gas." There were a number of books written on the subject of this "gas" and it was naturally seized upon as an explanation of various diseases of city dwellers, although the relation between the two was difficult to perceive. The result of the offensiveness of the air in many sewers was the practically universal use at one time of main traps between the sewers and the plumbing systems in buildings. The presence of these traps resulted in the impossibility of ventilating the sewers through the soil pipes within the buildings.

In some cases, however, ventilation was afforded by a pipe run up from the house drain, just outside the main trap, and generally carried above the roof on the outside of the building, although this position was often impracticable and substitutes were made for it, one of the worst being to have the ventilating pipe terminate in the "area" in front of the house, a foot or two above the ground. Many other methods of ventilating the sewers were also tried. One of the most obvious, which is still extensively employed, was to use perforated covers for the manholes. At one time perforated trays of charcoal were placed in the shafts of the manholes, in the belief that the sewer air in passing through them would be disinfected. In order to increase the draft up the vent pipe on the faces of the buildings, many kinds of cowls to surmount them were invented. Some of these risers were provided with a bent pipe admitting fresh air to their interior in a vertical direction, with a gas jet in the center of the vertical portion of this inlet, so that the flame of the jet drew a current of air constantly into the riser and also helped the upward draft in it from the sewer. Ventilating street lamps have been installed, particularly in British cities, in which the air is drawn from the sewer in a pipe and sucked up a shaft resembling an ordinary gas lamp post, by the draft of a gas lamp, through the flames of which the sewer air must pass before it can escape.

With the steady improvement in the construction of sewerage systems and the abandonment or rebuilding of the old lines which were defective, the annoyances due to foul odors became so rare that it occurred to many engineers about the same time that the necessity for main traps no longer existed where the sewers were in good condition, and that the ventilation of these sewers would be greatly helped by the omission of such traps. This opinion led to a number of investigations of the real nature of sewer air. One of the first of these was made by J. Parry Laws at the direction of the London County Council. He found that the bacteria in the sewer air were related to those in the external air and not to the bacteria of sewage. The inference he drew from this was that no matter how many germs of disease might be in the sewage they were not likely to enter the air above it unless the sewage splashed violently, as would be the case at the entrance of a branch sewer into

a trunk sewer at a considerably different elevation, or where the sewage fell down a manhole shaft. There was little probability, in his opinion, of bacteria passing from the walls of a sewer to the air, after the sewage level had fallen, because he found in one experiment that an empty pipe sewer, to which large numbers of bacteria must have been attached, effected no increase in the bacteria in a current of air sent through it. Although his experimental evidence was contrary to the probability of sewer air containing disease germs not found in external air, he nevertheless drew the following conclusions:

"Although one is led almost irresistibly to the conclusion that the organisms found in sewer air probably do not constitute any source of danger, it is impossible to ignore the evidence, though it be only circumstantial, that sewer air in some cases has had some causal relation to zymotic disease. It is quite conceivable, though at present no evidence is forthcoming, that the danger of sewer air causing disease is an indirect one; it may contain some highly poisonous chemical substance, possibly of an alkaloidal nature, which, though present in but minute quantities, may nevertheless produce, in conjunction with the large excess of carbonic acid, a profound effect upon the general vitality."

In 1907 Dr. W. H. Horrocks found at Gibraltar that where sewage fell vertically the air in the sewers contained the colon bacillus and various streptococci. He also found that it was possible to put easily recognized forms, such as B. prodigiosus, into sewage and recover them from the air of the sewers, into which it was assumed that they entered by the bursting of bubbles of gas rising from the sewage, from splashing of falling sewage, or from the drying of the sewage left on the walls of sewers when the depth of flow dropped. Other experiments of the same nature were made about the same time by Dr. F. W. Andrewes, and were recorded in the report of the Medical Officer of the Local Government Board for 1906–07.

Prof. C.-E. A. Winslow found in 1908, in an investigation for the Master Plumbers Association of Boston, that while the results of the investigations of Horrocks and Andrewes were undoubtedly correct qualitatively, the number of bacteria thrown off from sewage was so extremely small that the local infection of the sewer air was of no importance whatever. The general air of the house drains was found to be singularly free from bacterial life. Even near the points where splashing occurred there were only four times when intestinal bacteria were found, which led Prof. Winslow to conclude that, so far as infection is concerned, sewer air is not to be held responsible for the spread of infectious diseases.

It is the general opinion of engineers today that when a sewerage system is well designed, carefully built, and properly maintained, the sewage passes from the houses to the disposal works or outlets in a steady course which affords little opportunity for the subsidence of suspended

matter or the occurrence of offensive putrefaction and fermentation. Unfortunately accidents occur through the breaking of the crowns of pipe sewers, the settlement of heavy masonry sewers, and other misfortunes, which may cause sewage to collect in pools or at least to lose velocity to such an extent that more or less of the solids will settle to the invert. When this happens the sewer is likely to eventually become offensive. It follows from this that the maintenance of a sewerage system should always be well provided for, and those in charge of the work should appreciate the importance of investigating every complaint which is made regarding foul air from the system. These disturbances of the proper operation of the sewer network are generally considered as the only excuse for retaining any longer the main traps on house connections, which it is now believed by most engineers are the main obstacle to the efficient ventilation of sewers without recourse to the various devices and expedients of an earlier date. In other words, the recent improvements of sewerage systems, effected by a small expense for more complete engineering planning and more rigid supervision of construction, have saved a considerable expense in ventilating appliances and a great deal of annoyance to property owners on account of disagreeable odors. Prof. Winslow stated in 1909, in a letter read before the Boston Society of Civil Engineers:

"While we are right in spending money for plumbing which is free from gross defects, we are not as obviously justified in recommending large expenditures for refinements like back-ventilation and intercepting traps between the house and the sewer. The trapping of ordinary fixtures does away with most of the possible dangers of sewer gas. There are plenty of traps which will give a reasonable degree of security against siphonage without back-ventilation."

One of the best proofs that these conclusions are correct is the fact that the laborers engaged continually underground in the sewers of Paris, are kept under strict observation and there is no indication whatever that their work in sewer air has any effect on their health.

There are a few authentic cases of loss of life in this country due to sewer air.[1] One of these is mentioned on page 552, and occurred near the outlet of the Los Angeles outfall sewer. Another happened in a gate chamber of the intercepting sewerage system in Syracuse and a third happened in 1906 in a dead end in San Francisco. In each case it is probable that the gases given off by the changes in the composition of the sewage, which are usually carried along within the sewage to a certain extent, were liberated from the sewage and collected at the places where the accidents occurred. The actual composition of the gases is unknown,

[1] There have been a few cases where death was apparently due to the accumulation of illuminating gas in sewers.

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Many complaints of foul odors were made and finally led to adopting a policy of closing the manhole covers on ascertaining that odors actually came from them, and running ventilating pipes up the adjoining building if permission to do this could be obtained. This was in accordance with the practice of many other English cities. In order to ascertain how much circulation was really obtained through these manhole covers, Mr. Mawbey carried out many experiments. In a typical instance a 6 × 4 in. shaft was erected between two manholes 150 ft. apart. Anemometer tests showed that in both manholes the outlet currents of air, after the shaft was erected, exceeded the inward currents in the proportion of 69 to 20 in one case, and 41 to 19 in another. In another case where a complaint was made of odors at a manhole at the intersection of two large sewers, two stacks of 9-in. stoneware pipe were erected side by side and the manhole cover left open. Anemometer tests showed that the upward current of air with the double shaft was 505,000 cu. ft. per day, and although it was only about 94 ft. distant from the manhole cover, there were upward currents through the latter of 40,500 cu. ft. a day, while the inward currents were only 18,000 cu. ft. per day. The cover was still a nuisance and was closed. Many similar experiments were made, which showed that the column of air in the manholes was too low to make the ventilation through their covers a matter of any importance. This confirms the general American opinion that it is best to ventilate the sewers through the house connections, when the sewerage system is in good condition and there are good plumbing regulations which are enforced strictly.

The authors have found that many complaints of offensive odors from sewers have been due to the discharge into them of industrial wastes, such as refuse from gas works. In one case, the trouble was traced to crude oil, which had escaped from the underground piping of a forging plant and percolated into the sewer through leaky joints. Packing house refuse is particularly offensive, and if discharged into a sewer having a sluggish current it may be the cause of foul odors. Many times objectionable odors are forced out of perforated covers of manholes by steam discharged into the sewers. In fact, odors are more likely to be given off from hot than from cold sewage.

CHAPTER XVII

SEWAGE PUMPING STATIONS

In the design of sewerage works, it may be necessary to resort to pumping where the sewage or storm water is collected at so low an elevation that discharge by gravity is impossible, as at Washington; to reach a desirable purification site, as at Baltimore; to lift the sewage from areas too low to drain into the main system by gravity, or to force water into streams or tidal inlets receiving sewage, which would become offensive unless flushed in this way.

Whether the sewage shall be lifted at one or more points is usually a matter to be settled by comparing the fixed and operating expenses of different plans. The operating expense of raising all of the sewage at one point is less than that of doing this at two or more points. On the other hand, if all of the sewers are made to drain by gravity to one place, their cost may be greatly increased on account of the deep cuts and large cross-sections necessary in order to obtain satisfactory velocities of flow. Various projects must often be considered, both with and without pumping, and the extra cost necessary to drain to one point, together with the cost and the capitalized annual charges for operation and depreciation of the pumping station, must be compared with similar charges for a project with two or more stations. Conditions may even arise where, if the 24-hour flow can be handled by working the station at its most economical rate for 8 hours, the reduction in labor charges accomplished in this way will warrant the construction of reservoirs to store the sewage when the pumps are not running. The trunk sewers of combined systems sometimes have such a large capacity that they afford considerable storage capacity during dry weather.

Comparison of Different Designs.—This matter of the relative economy of different designs is not so simple as it appears at first thought, but involves a number of factors. Prof. Geo. F. Swain gave in the "Journal of the New England Water Works Association," vol. ii, p. 32, the following as the correct manner in which to attack the subject:

"The problem, in its most general form, may be considered to be this: a certain structure or machine costs A dollars, it requires the expenditure of B dollars for repairs at intervals of s years, it will last for n years, and when worn out it may be sold for D dollars. A second structure or machine for accomplishing the same object costs A_1 dollars, requires the expenditure

of B_1 dollars for repairs every s_1 years, lasts for n_1 years, and is worth D_1 dollars when worn out. Which of these will be more economical, as a permanent thing, the rate of interest being r, payable semi-annually?

"To answer this question we must compute the amount of present capital sufficient to provide permanently for each of these structures, and the one which requires the smaller capital will be more economical. Or we are enabled to find, by the same method, what the cost A_1 of a (perhaps new) appliance must be, in order that it may be more economical than a similar appliance in use, under various suppositions as to the life, cost of maintenance, etc.

"The present capital required for any structure will be made up of three parts.

"First, A, the cost of the structure.

"Second, a sum which, put at interest at r per cent., will increase in s years, by the amount B. This sum may easily be shown to be

$$\frac{B}{[(1 + 0.5r)^{2s} - 1]}$$

r being expressed as a proper fraction (6/100 if the rate is 6 per cent.).

"Third, a sum which, put at interest at r per cent., will amount, in n years, to itself plus $(A-D)$; since at the expiration of the n years, the worn-out structure being sold for D dollars, there will result a sum sufficient to again expend A for a new structure, and have the original sum remaining, which in another n years will amount to sufficient to purchase a third structure, and so on indefinitely. This sum is

$$\frac{A - D}{[(1 + 0.5r)^{2n} - 1]}$$

"The total present capital involved in the use of any structure is therefore

$$C = A + \frac{B}{[(1 + 0.5r)^{2s} - 1]} + \frac{A - D}{[(1 + 0.5r)^{2n} - 1]}$$

"In certain cases the formula is simplified. Thus if D is so small as to be practically zero in comparison with the first cost of a new structure, and if B is the uniform annual cost of maintenance (supposed payable semi-annually) as in the case of a pumping engine, we have

$$C = A + \frac{B}{r} + \frac{A}{[(1 + 0.5r)^{2n} - 1]}$$

in which A is the first cost of the structure or machine, and B/r is the capitalized cost of maintenance. This result shows that it is not strictly correct, in comparing, as a permanent investment, let us say, two pumping engines which may be supposed of equal durability, to compare simply the first cost plus the capitalized cost of operation, since this omits the last term in the above formula. This term, however, when n becomes large, rapidly decreases, and in many cases may well be neglected."

STORAGE AND SCREENING

In most sewage pumping stations there is provision for some storage of the sewage, in order to equalize the fluctuations in the rate of flow

through the sewers sufficiently to enable the pumps to work properly. In a few cases this storage is extremely limited and it would be impossible for the pumps to run, except for a few minutes at a time, were it not for unusual steadiness in the rate of flow to the station, something which the designing engineer should be very slow to place any reliance upon. Where the basins are used simply to improve the load on the pumps, and the amount of dry weather sewage is large enough to keep a fairly economical pump running under a good load, there seems to be a tendency at present (1914) to rely very little upon storage and to provide enough pumping units, thrown into service one after the other, so that there is only one unit running on a poor load at one time. This is accomplished in small stations by automatic controllers which start the first pump when the sewage level reaches a certain height, another one when it reaches another height, and a third when it reaches still a higher elevation. The very short time when the second and third pumps are in operation make their performance a matter of no importance so far as economy is concerned, reliability and low cost being the most essential items to be considered.

The operation of the pumps can be kept under observation at any point by means of the Winslow apparatus, used quite extensively to record the height of water in reservoirs. An apparatus of this sort at Lynn, Mass., for instance, where there is a three-unit pumping plant with automatic starting and stopping devices, shows at the office of the Lynn Electric Department the height of sewage in the wet well at any instant and the time of starting and stopping the pumps. At Waltham a Winslow recording apparatus, with dials and weekly record of pumping, was installed both at the office and in the foreman's house at the sewer yard.

The storage basins in connection with sewage pumping stations are usually merely small wet wells,[1] for with the separate system the amount of storage required is small and with the combined system the sewers themselves have considerable storage capacity during dry weather, when the flow must be retained for varying periods in order to provide a proper load for the pumps, and during storms there is no necessity of storage since a suitable load for the pumps exists continually. The shape given to these wet wells depends very largely on the general arrangement of the pumping machinery, as is shown by the illustrations of typical pumping stations given later in this chapter. In working out the details of such a well, the designer should provide some system of positive ventilation, because sewage is likely to carry gases with it

[1] Pump wells are termed "wet" when sewage flows into them, and "dry" when the sewage is carried through them in pipes or closed conduits. If a submerged centrifugal pump is used, it may be placed in a wet well, or the same operating conditions can be obtained by placing it in a dry well at such a depth that its suction pipe will deliver sewage under a head from a wet well.

which are given off during storage; illuminating gas has been known to escape into sewerage systems, and of late gasoline entering the sewers from garages has become volatilized and caused some explosions.

One of the few comparatively large sewage regulating basins in this country was built in 1899 at Concord, Mass., from the plans of one of the authors. Its purpose was to store the flow of the sewage during hours when an electric lighting plant operated in conjunction with the sewage pumping station was carrying its heaviest load, and to give better distribution upon sand filter beds located at the end of a long cast-iron force main. This well has an internal diameter of 57 ft. and a storage capacity of 222,000 gal. It has brick walls from 20 to 24 in. thick, an inverted parabolic groined arch bottom of concrete, and an elliptical groined arch roof of concrete, with 24-in. brick piers 14 ft. 9 in. on centers. The construction of this well was extremely difficult and was described in the "Journal" of the Association of Engineering Societies, May, 1900.

Another reservoir of the same type was built in Clinton, Mass., by the Metropolitan Water Board in 1898. It has an inside diameter of 100 ft. and a height of about 13 ft. The roof is supported by concrete groined arches and brick piers 14.57 ft. apart on centers. The side walls are 2 ft. thick at the top and 3 ft. 6 in. thick at the bottom. The bottom and roof each have a minimum thickness of 12 in. The trunk sewer from the city terminates in a screen chamber between this reservoir and the pumping station, which are close together, and the sewage can be turned into the reservoir and given an opportunity for a large amount of sedimentation or it can be sent directly to the wet well from which the pump suction runs.

Both these structures have groined roofs, a form of construction which is incapable of satisfactory mathematical analysis, although it has been used for so many years that practical experience has shown that certain general dimensions can be employed safely. A discussion of the methods of designing such roofs, by Thomas H. Wiggin, was printed in *Eng. News*, April 7, 1910, and as the different methods are all empirical none of them should be used without a study of this article, in which their limitations are pointed out and important data concerning groined arch roofs are tabulated.

There have been a number of failures of groined arch roofs in the United States, and a lack of confidence in them is felt at the present time (1914) by some engineers. They have certain advantages, however, which the designer should carefully consider before adopting another form of construction. In the first place, their first cost is usually no greater than that of reinforced concrete slab roofs, as is pointed out in detail in the article by Mr. Wiggin to which reference has been made. In the second place they are free from steel, either exposed or enclosed

in concrete. This is an advantage, although it is difficult to give it any definite value. In the winter the sewage may be considerably warmer than the outside air, and it is entirely possible that the roof and walls of the reservoir at such a time will be reeking with moisture. Ordinary moisture is injurious to steel and it seems probable that the moisture in a sewage reservoir may prove still more destructive. Experience has indicated in Boston, for instance, that under certain conditions the metal in sewage wells is liable to become seriously corroded. Such experience has been observed elsewhere, but observations are so discordant that the only safe deduction from them is that the atmosphere in a sewage reservoir is likely to be particularly severe in its action on steel, which makes the use of I-beams and reinforced concrete unusually expensive on account of the necessity of using exceptionally large amounts of metal.

Except in small plants, provision is usually made for screening, and sometimes for sedimentation, of the sewage before it reaches the pumps. There is no uniformity of opinion among engineers regarding the size of screens, either as to size of bars or size of openings. In fact where sewage comes from an industrial district where rags and waste are likely to be thrown into it in large quantities, some engineers believe that it is less expensive in the end to install a relatively large automatically controlled pump which can successfully handle unscreened sewage than to use a smaller pump which makes screening a necessity, as such screening involves much more or less continuous labor charges. Experience seems to indicate that such sewage may be handled without screening by pumps 8 in. or larger in size and that smaller pumps are likely to become clogged more or less frequently, the trouble increasing as the size is reduced.[1]

The screen chamber of the Old Harbor Point pumping plant of the main drainage system of Boston, which was officially put in service on Jan. 1, 1884, is a structure independent of the pumping station. It is 25 × 32 ft. in plan, inside measurement, and the 10-1/2-ft. main sewer terminates at one of its end walls. There are two channels in the bottom of the chamber, leading from the inlet to openings in a transverse wall, each opening being closed by a 7 × 6-1/2-ft. sluice gate. On the back of the opening in the wall there is a screen cage, 7 ft. 8 in. high, 7 ft. 3-1/2 in. wide, and 3 ft. 4-1/4 in. deep, with the back, sides and top formed of 3/4-in. round iron rods with 1-in. spaces between them. These cages are

[1] The opinion is sometimes held that on account of its open passages and absence of valves a centrifugal pump will handle anything which will pass through the suction pipe. This is not universally true. Experience with the sewage pumps at Newton, Mass., showed that cotton waste entering the sewers from large machine shops collected about the suction pipe and, when finally drawn in, stopped the discharge completely. Basket screens were placed at the inlets of the pump well and stopped the trouble; the mesh was quickly clogged and the sewage was forced to overflow the rim of the basket, but the waste was intercepted as if the sewage passed through a sump.

counterbalanced and raised or lowered by small steam engines. Back of this transverse partition is another, also provided with two openings with screen cages, but without sluice gates. There is a longitudinal central wall running from the rear end wall of the building and intersecting both transverse walls, so that in plan there are four screen pits and a main extrance pit. The screens were made in the form of cages in the belief that they would retain the solids while they were lifted to be cleaned, thus making that operation easier, it was hoped, than cleaning an inclined or vertical rack.

Recent designs for a sewage pumping station in Boston have called for inclined racks making an angle of about 30 deg. with the vertical, and the designer, C. H. Dodd, informed the authors that he would use a still flatter slope when practicable on account of the greater ease in raking the screens. Where vertical screens are used, some engineers attach a horizontal ledge or trough to their bottoms to catch any material which may drop from the bars when the screens are raised.

The screens used in the Boston Metropolitan sewerage plants are cages formed of iron frames 8 ft. 3 in. high, 5 ft. 4 in. wide, and 2 ft. 11 in. deep, held in position by guides. The front of the cage is open. The back and two ends are double rows, staggered, of 3/4-in. rods spaced 1–3/4 in. on centers. They are raised and lowered by double drum steam hoisting engines.

The sewage pumping station built in Detroit in 1912 has an independent screen chamber on the 9-ft. sewer running to the main building. It is nearly circular in plan, the deviation from a circle being due to a flattening of the walls so as to produce straight sides where the pairs of screen guides are located. There are two sets of screens, one 3 ft. behind the other, and the chamber is 15 ft. wide where they are located. Each screen is 10 ft. high and 7–1/2 ft. wide, with 3/4 in. bars 12 in. apart. The bottom of the screen has a horizontal shelf to catch trash. The center guide for the screens is a pair of channels placed back to back, and the side guides are made of *Z* bars with one leg imbedded in the concrete wall. The screens are counterbalanced and raised by hand. The exterior of this chamber is shown in Fig. 323.

Special provision for sedimentation is rare where centrifugal pumps are used, but an unusually good example of it is afforded by the chamber built for that purpose at the sewage pumping station in Washington, illustrated in Fig. 321.

At the Colombes pumping station of the Paris sewerage system, the initial (1894) equipment, which consists of reciprocating pumps, had a total capacity of 31,700 gal. per minute, and to prevent injury to their water ends a settling basin of 3229 sq. ft. area has been constructed. It was described as follows by Bechmann and Launay in the "Annales des Ponts et Chaussées," 1897.

"The sewage is discharged into the basin to free it from foreign bodies, sand and grease. At the inlet, which is in direct connection with the outfall sewer (Aqueduc d'Acheres), there is a screen of 128 bars 0.158 in. thick, making an angle of 22 deg. with the vertical and 0.8 in. apart, center to center. Between alternate pairs of bars move the teeth of eight rakes, which have a uniform speed of 3.9 in. per second around the external frame of the rack. The teeth of these rakes, gathering the refuse, remain horizontal up to the moment of unlatching, which occurs automatically when certain roller cams escape from their guides. The rakes are moved by tackle driven by a 1-h.p. electric motor.

"Immediately behind the rack is the basin. It is rectangular, 98.4 ft. long, 32.8 ft. wide, with its floor about 4.9 ft. below the invert of the sewer at the inlet. It has a concrete bottom and masonry walls, and all surfaces with which the sewage comes in contact were given a coat of Portland cement plaster. On the sides and lower end of the basin there are weirs, with flashboard regulation, which enable the discharge to be adjusted to correspond with the rate of pumping. The checking of velocity due to the great length of the weirs, 213 ft., and the large capacity of the basin result in the precipitation of the solid matter.

"To remove this solid matter, composed for the most part of organic substances, in a continuous manner a very satisfactory device (resembling a clamshell bucket) is employed. It is constructed of two half-cylinders of steel plate, able to oscillate about a common shaft, provided along the lower sides and ends with teeth which cross each other at the time of closing the apparatus. Cranks, shafts, chains and latches are provided for automatically opening and closing the apparatus. The device holds about half a cubic yard. It is mounted on a timber frame moved by electric motors. The dredged material is placed in small cars, which are emptied beside the railway on which the material is moved, whence the farmers remove it as fast as it is delivered."

The subject of screening as a method of sewage treatment is discussed in detail in Volume III.

PUMPS

The equipment of a pumping station must be selected with a view to meeting the usual working conditions in the most economical manner which the funds available for the plant permit, and also to meeting the maximum conditions with such a degree of efficiency as is warranted by the frequency of their occurrence. A great mistake is made by the purchaser of a pumping plant which will operate with the highest efficiency only when subject to its very infrequent maximum conditions or something approaching them. The late Charles A. Hague mentions a case of this sort in his "Pumping Engines for Water Works:"

"The engines advertised for were proportioned, according to the specifications, to pump against a head 50 per cent. greater than really developed in regular service, with the result that triple expansion engines were placed

under conditions where compound engines with smaller steam ends would have undoubtedly done much more economic work. What happened, apparently, was that the high and intermediate pressure cylinders did so much of the work that there was only a low-temperature fog left for the low-pressure cylinder to handle, and the third plunger was largely operated through the medium of the crank and connecting rod, dragging the low-pressure piston along incidentally."

In the selection of pumping plants for sewerage work the primary consideration should be reliability of service. This means not only sturdy construction but also, in the case of electrically operated pumps, reliability of sources of current. First cost should never be considered by itself, but only in connection with operating charges, for the total of the annual fixed, operating and depreciation charges is the item to be studied. Floor space must sometimes be regarded as important, and less frequently the desirability of having the water end at low levels, of starting and stopping the pumps automatically, or of combining the sewage pumping plant with a refuse incinerator, after the system occasionally used in England, or a lighting station, as at Concord, Mass.

In estimating costs, it is desirable to obtain actual operating costs from places using plants like those under consideration. It is inevitable for manufacturers to state the steam or current consumption of their pumps as small as they consider it safe to place them, while the engineer must be more liberal. If the capacity of a plant is made close to the actual needs and then it is bought under a guarantee of performance, any failure to equal that performance can rarely be made good by a deduction from the contract price; the plant will not perform its service as it should, thus perhaps throttling the sewer system, and the engineer has failed to properly safeguard his client. This is particularly important in connection with centrifugal pumps, for experience has shown many times that their capacity has not equalled that contemplated in the plans. It is often false economy to curtail the cost of a project by paring down the size of a pumping plant.

The engineer must also be very cautious about using the information regarding the cost of sewage pumping given in annual municipal reports, for very often the pumpage is actually unknown and the quantity reported merely a guess. The pumps run under operating conditions which would not exist in stations of more modern design, and the charges for attending to screening are lumped with those for running the pumps, which may cause considerable error if the conditions are like those at Lynn, at one time, where the cost of keeping the screens clear is reported to have been greater than the cost of pumping.

In comparing plants financially the first cost of the complete plant of each type and size should first be estimated, from which the annual fixed charges can be ascertained. Then the operating and depreciation expenses for the present steady load should be estimated, and also those

for the extra loads and for the steady load some years later. In small plants such estimates generally indicate, in connection with local conditions and the engineer's opinions regarding reliability, that the choice will be between a few sizes of one type, which should then be studied in detail.

In this connection it should be pointed out that sometimes the construction of a small pumping plant for temporary service is preferable to the immediate construction of an expensive sewer too large for any use that will be made of it for a number of years. This was shown at Newton, Mass., where a station costing $6700 and designed for a useful service of only 10 years, was $500 a year cheaper for that period than the fixed charges on the alternative, a sewer costing $45,000.

The relations between the reservoir, pumping and force main capacities were stated as follows by Frank A. Barbour, in a discussion on small pumping plants for sewage, before the Boston Society of Civil Engineers, Jan. 9, 1907:

"The more nearly continuously a pumping plant can be made to operate at a uniform rate, the more economical is the result, provided the labor account does not offset the saving in a decreased storage capacity, lessened size of force main and reduced friction head thus made possible. With a steam plant a reservior large enough to hold the sewage during the hours the pumps are not running and a force main adapted to the pump rate are required. If the amount of sewage to be handled could be accurately predicted and a pumping unit equal to the average daily discharge adopted, then a reservoir only large enough to equalize the hourly variation would be necessary. This cannot practically be done and the nearest approach to uniform discharge is obtained by dividing the plant into such a number of units as will most nearly approximate in their capacity the rate of inflow; in other words, by dividing the total power into units best capable of handling the load curve. With such an arrangement, the reservoir can be reduced to a size only sufficient to prevent too frequent starting and stopping of the units, and the force main can be designed upon the basis of the maximum rate of inflow for the period in the future which it is economical to consider. This continuous discharge is often extremely desirable in disposal works, where either purification is effected or the sewage disposed of by dilution."

As a general proposition in designing pumping stations, it is advisable to determine as accurately as possible the work required constantly from the first day of operation, then estimate the additional load which will arise from time to time on account of emergencies for which provision must be made in the original installation, and finally estimate the requirements for such future time as seems desirable. In this way the engineer will be able to adopt units of such size that one or more of them will be handling the ordinary load with good economy and the charge for readiness to meet emergencies will be restricted to the fixed charges on the equipment for the purpose and will not be swelled by the losses incident

to operating equipment of large capacity under loads which make its performance highly wasteful.[1]

The selection of the pumping equipment should be governed by a regard for operating and maintenance charges and facility in making probable extensions in the future, as well as by first cost. The time spent in analyzing the needs and the methods of meeting them should never be skimped, for if a pumping station is required it is particularly desirable for it to be a good one, because its reconstruction without interfering with service will be troublesome and if its operation is unsatisfactory the whole sewerage system is burdened with a defect. The station is not an isolated detail, but an integral part of the system.[2]

It is not necessary in this place to go into the details of the design of pumps[3] or the arrangement of steam plant, as these specialties require a large amount of space for thorough treatment, and are well covered in a number of books. The following notes are intended merely as a schedule of the points to be considered in working up the outline of a pumping plant, and as a guide to the steps to be taken in making a final selection of the equipment.

RECIPROCATING PUMPS

Most reciprocating pumps used in pumping sewage are driven by steam, although the triplex type, which is well adapted for this service,

[1] An early illustration of this principle will be found in the pumping station of the Boston sewerage system, which is explained in the report by Eliot C. Clarke (1885) as follows: "As the city sewers receive rain-water, and as it is desired to take as much of this as possible, especially from certain districts, it follows that during short periods of time, when it rains, very much greater pumping capacity is needed than is usually sufficient. There must, therefore, be a pump, or pumps, to run continuously, and others to run only when it rains or thaws. The chief item of expense in pumping is the cost of fuel. For the sake of economy the pumping engines for continuous service must do their work with as little consumption of fuel as possible, and to accomplish this an expensive machine can be afforded. For the engines which run only occasionally cheaper machines are more economical, the saving in interest on the first cost more than compensating for the extra fuel consumed by them. The pumping plant of the Boston main drainage works includes two expensive high-duty engines and two cheaper lower-duty engines. The high-duty engines were designed by E. D. Leavitt, Jr., on general specifications prepared by the city engineer, Mr. Davis. They were built by the Quintard Iron Works, of New York, and cost about $115,000 each; nominal capacity 25,000,000 gal. each. The two pumping engines for storm service were built by the firm of Henry R. Worthington from their own designs and cost $45,000 each. They are of the duplex, compound, condensing type." The larger engines developed 122,000,000 and 125,000,000 ft.-lb. per 100 lb. of coal on trial, while the Worthington engines were guaranteed to deliver 60,000,000 ft.-lb.

[2] A detailed explanation of the methods followed in selecting the most economical motive power for pumping sewage at Lynn, Mass., is given in *Engineering and Contracting*, Jan. 7, 1914, by Frank H. Carter.

[3] For information regarding large steam-driven pumps, the reader is referred to C. A Hague's "Pumping Engines for Water Works," for a description of all types of pumps to Prof. A. M. Greene's "Pumping Machinery," for centrifugal pumps in particular to Fritz Neumann's "Die Zentrifugalpumpen" (Berlin, Julius Springer) and C. G. de Laval's "Centrifugal Pumping Machinery," and for the design of steam plant to "Steam Power Plants," by Henry C. Meyer, Jr.

is as well suited for electric motor or gas-engine operation as for steam operation. The smaller triplex pumps for sewage are usually provided with ball valves and the larger sizes with leather-faced clack valves. As a matter of fact, a power pump driven by a direct-connected motor or engine or through the medium of a noiseless chain or even a belt may be the best equipment for certain conditions. The pump can be located at a lower level than the engine, if desirable, and the economy of some of the small steam and internal combustion engines now available is much greater than that of direct-acting pumps of all but the larger sizes with so-called triple expansion.[1]

Types.—There are two types of steam pumps, the direct-acting and the flywheel. In the direct-acting type the steam cylinder is in line with the water cylinder operated by it and there is no flywheel to store up and give out energy. In the flywheel type the essential feature is a revolving flywheel which equalizes the angular motion of the shaft on which it is mounted and thus carries the engine past the dead centers at the ends of the strokes; in most flywheel pumps the plungers are driven by rigid connections with the steam piston rods, and the crank shaft and its flywheel are driven by a connecting rod or rods from the crossheads of the engine.

The direct-acting pump, having no flywheel and operating against an inert load, must take steam for the full length of the stroke in most cases, and there is no expansion in the ordinary sense of this term. At the end of the stroke, the steam valve is thrown in a variety of ways, each pump maker having a special type of gear for the purpose, and steam pressure is admitted to the other side of the piston. In a so-called compound direct-acting pump the exhaust from the high-pressure cylinder goes into a low-pressure cylinder for similar use throughout the whole stroke, and in the so-called triple-expansion direct-acting pump, the exhaust from the intermediate cylinder goes into the low-pressure cylinder for a full stroke. In this last type the total expansion of the steam is rarely over seven or eight times, much less than the expansion in a flywheel pump with a steam end operated like a standard engine for power service. As a result of the small number of expansions possible in a compound or triple direct-acting pump, it is unnecessary to use as high a steam pressure with it as is desirable with a compound or triple-expansion flywheel pump. On the other hand, good condensers and steam jacketing are held by most designers to be particularly useful with the larger direct-acting pumps.

The duplex direct-acting pump consists of two complete pumps side

[1] "As to pumps driven by a belt from a Corliss or other high-economy engine, this arrangement undoubtedly offers for small plants higher economy than can be had with direct-acting pumps of the ordinary type, and the writer has installed many engines of this general class, although usually driving the pumps by gearing rather than by belts." Irving H. Reynolds, *Trans. Am. Soc. C. E.*, vol. liv, p. 604.

by side, with the main steam valve of each operated by a connection from the crosshead or an equivalent reciprocating part of the other. Such a pump is remarkably self-contained and can be manufactured of good design and materials at such a low price that it soon became the leading type for most purposes where moderate quantities of water had to be handled. Since the great improvement in centrifugal pumps its field has been somewhat curtailed, but its reliability, demonstrated by many years of varied service, makes its consideration necessary in selecting the equipment for a great range of service.

The duty of such pumps in water-works service is usually guaranteed at about 60,000,000 ft.-lb. per 1000 lb. of dry steam for compound condensing and 90,000,000 ft.-lb. for triple condensing units. Small sizes will not show anything like such duties in operation, however, the range being from 10,000,000 to 40,000,000 ft.-lb. or from 198 to 50 lb. of dry steam per actual horse-power per hour, depending on the size, operating condition, and ratio of load to capacity.

High-duty attachments can be added to large engines of this type, which will make them much more economical, but they are hardly required in sewage pumping work.

Where the capacity of the pump is from 3,000,000 to 5,000,000 gal. a day and high duty is required, a compound flywheel pump has advantages which must be considered. This type has steam cylinders with cutoff valves, and its simplest form is a horizontal cross-compound engine with a pump cylinder tandem to each steam cylinder. The Holly-Gaskill pump much used in water-works service is a more elaborate form consisting of two compound engines connected to the same crank shaft, and a pair of water cylinders with double-acting plungers. The Snow compound pump is a later and less complicated type, and while it occupies more space than the Gaskill its parts are more accessible and the power is transmitted more directly.

The guaranteed duties of horizontal compound condensing flywheel pumps are usually from 110,000,000 to 130,000,000 ft.-lb. per 1,000 lbs. of dry steam.

Horizontal flywheel pumps have occasionally been built for triple expansion, but the large floor space they occupy in comparison with vertical pumping engines usually renders them less desirable than other types.

Vertical pumping engines usually follow the general design worked out first by Edwin and Irving H. Reynolds. These are usually triple-expansion, and as inquiry is often made why a compound condensing steam end is not more often used, the following statement of the reason is reproduced here from a paper read before the International Engineering Congress of 1904 by Irving H. Reynolds (*Trans. Am. Soc. C. E.*, vol. liv–d, p. 519).

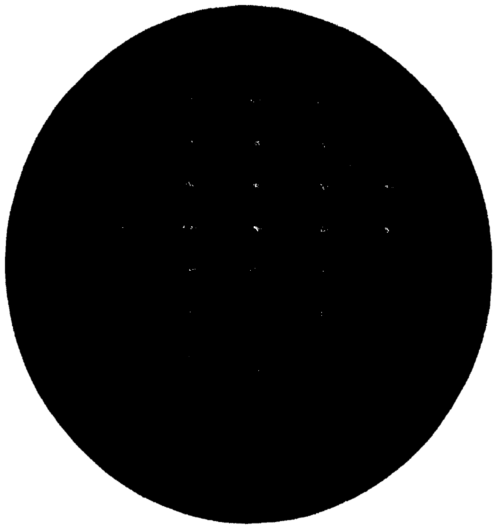
"It has been argued that with the low steam pressure often used, a compound engine would give practically the same economy as the triple and at much less first cost. While this is to some extent true, the fact is overlooked that economy is not the sole reason for the adoption of the triple, but that the general excellence of the triplex pump for handling water and the adaptability and flexibility of the machine as a whole are the factors which are responsible for its wide popularity. Having determined on three single-acting pumps as the best and simplest form, it is essential, in order to drive them direct, to have three steam cylinders, and thus there is obtained the triple-expansion engine, practically without increased cost and with a steam economy from 10 to 20 per cent. higher than that of a compound engine working under similar conditions."

The guaranteed duties for triple-expansion pumping engines vary from about 140,000,000 ft.-lb. per 1000 lb. of steam for small units and moderate steam pressures to 160,000,000 ft.-lb. for large engines and steam pressures of 150 to 175 lb.

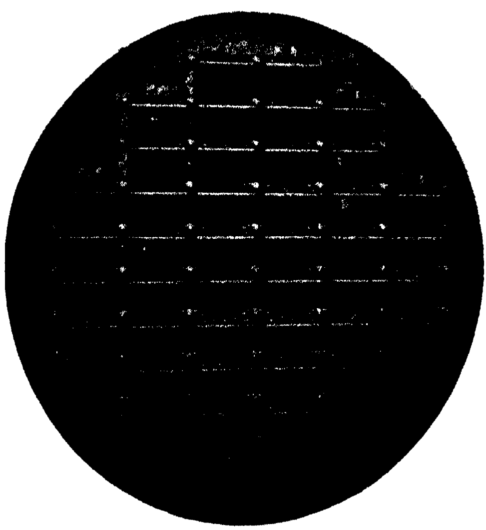
Piston Speed.—There is considerable discussion at present regarding the proper piston speed of these large engines. In Mr. Reynolds' paper, already quoted, it is stated that speeds higher than 200 to 250 ft. per minute probably offer no advantage, because the small clearances of the slow-speed engine enable it to show as high economy as the high-speed engine, despite the theoretical advantages of the latter. Furthermore, with high speed, the cost of the water end is, if anything, increased, for as the time allowed for the seating of the valves is less, more area must be provided, and to avoid friction losses all ports and passages must be maintained fully as large as on slow-speed pumps. In the discussion of the paper, C. G. de Laval presented the argument for piston speeds higher than 200 ft. substantially as follows: When water is once in motion it is not a question of speed in feet per minute, but of changes of plunger or rotative speed, and these changes do not affect any other part of the pump end except passages and valves, which always should be made amply large to allow a low velocity through them.[1] The high speed allows smaller moving parts, which are less cumbersome, more flexible and easier to handle than the large parts of slow-speed engines, and will also insure easier making and stronger shapes with less metal than can be found in slow-speed engines.

Water Ends.—There are two distinct types of water ends for pumps, the piston and the plunger. In the piston type, the water cylinder has a cylindrical barrel throughout the distance traversed by the piston, which is fitted with flax packing or metal rings so as to allow as little leakage as possible between it and the walls of the cylinder. The latter usually has a brass lining to reduce the friction and help maintain the

[1] A velocity from 3 to 3-1/2 ft. per second through valve openings is usually considered best in large pumping engines.



Valves closed.



Valves open.

Fig. 301.—Valve deck, Baltimore sewage pumps.

(*Facing page* 658)

tightness. In the plunger type, the plunger does not touch the walls of the cylinders, but passes through a stuffing-box or packing ring which prevents leakage. Its action is not that of a piston, forcing through a cylinder all the liquid in front of it out to the cylinder wall, but it displaces in the chamber into which it is forced an amount of liquid equal to that part of its volume which is thrust into the chamber, whence it derives its name of plunger. It is much less expensive to keep the water end of a plunger pump in good condition, particularly when pumping gritty liquids, than that of a piston pump, and consequently this type should generally be used for sewage, and special attention should be paid to the position and arrangement of the stuffing-boxes, for they will probably need more attention in a sewage pump than in one handling water.

There are two types of valves in general use in pumps, the disk and and the clack. The disk valve is usually of rubber or rubber composition, although leather was formerly much employed. It is not often that they are more than 4½ in. in diameter. They are usually held down on their seats by helical springs. If a valve deck will not furnish room for a sufficient number of seats, it is perforated with large orifices to which hexagonal or octagonal boxes are attached. These boxes greatly increase the area to which the disk valves can be attached.

Clack valves, which are generally used in sewage pumps, are flaps either actually hinged or attached to the valve deck so as to move as if they were hinged. In the latter case they are strips of rubber 5/8 to 1 in. thick, usually with a metal disk on the lower side somewhat smaller than the opening in the valve seat and a heavier and larger disk on the top to add weight. Clack valves frequently cause much trouble because sticks and rags are caught on their seats and hold them open. Ball valves are also used to some extent. In English pumps, the clack valves are sometimes made of very thick leather, such as that from the hippopotamus and rhinoceros. Hinged clacks are more often used now than the simple flap pattern; they have a leather or rubber disk held between metal plates, the top plate having an arm running sideways to a hinge connection with the valve seat. The valve deck and valves of the Baltimore sewerage pumps, built by the Bethlehem Steel Co., and described briefly later in this chapter, are illustrated in Fig. 301. As large clack valves are likely to cause pounding, they are sometimes provided with a small clack valve on their upper surfaces. They should only be used with pumps of slow and moderate speeds, as they are sluggish in action.

The clack valves of the Leavitt pumps of the Boston main drainage works, built in 1884, are rubber, and great difficulty has been experienced with them, due largely to the breaking of the rubber where it acts as a hinge. In the Ward St. station of the Boston Metropolitan system

the valves are hinged and swing on a manganese bronze hinge bolt; they have rubber and canvas seats which are bolted to brass plates.

The English views regarding reciprocating sewerage pumps are stated as follows in M. Powis Bale's "Pumps and Pumping."

"If sewage or sludge is pumped by steam, a long-stroke plunger pump is generally to be preferred to a piston pump for this duty, but many large single-acting lift pumps are also in use. It is important for the liquid to have as few reversals of its flow as possible, and that there be no complications in the passages or corners where solids can accumulate. The valves should be as large and free as possible, and readily examined; sometimes for this work the valve seats are made movable as well as the valves. Wrought-iron clack valves with leather seats are used for sewage purposes, and also double-beat valves. Sewage lift pumps are often made of cast iron, with leather buckets and valves, the clacks of leather weighted with iron plates."

The above statement regarding the reversals of flow and absence of pockets to collect sludge, is particularly important in connection with sewage pumping. It is true that any checking of velocity in a pump chamber will occur for such a short period that there is little opportunity for sludge to settle from sewage, but the less chance there is of this the smaller the probability of clogging in passages and the accumulation of leathery coats on the valve decks. On this account the pump details for handling sewage should be more open and direct than are sometimes considered necessary where clear water is handled; pumps of different makes are unlike in these details and as it is unwise to go to the expense of a special design for a small sewage pump, the different details of standard commercial pumps should be scrutinized carefully to ascertain which are the most suitable for sewage.

Connections.—The following suggestions regarding the connections of reciprocating pumps were issued by the Snow Pump Works.

"Faulty connections are generally the cause of the improper action of a pump, and great care should, therefore, be taken to have everything right before starting. To accomplish this, note carefully and understand thoroughly the following:

"Be sure that the quantity of water you desire to pump is available and that your pump is within easy reach of it when it is at its lowest level.

"Locate your pump as near the source of suction supply, both vertically and horizontally, as is possible or convenient; but never place it in such a location that the sum of the following three items will exceed a total of 26 ft. 1. Height in feet from the discharge valves of the pump to the lowest level of the surface of the suction water. 2. Total friction loss in suction pipe in feet head. 3. Total friction loss in feet head due to elbows and tees (assumed as being equivalent to the friction loss of 100 ft. of same size ot pipe, for each elbow or tee).

"Lay your suction pipe so that it slopes away from the pump gradually. A suction pipe should have no air pockets in its entire length, but should be so laid that if air be admitted to it, near the intake end, with the pump standing still, the air would rise to the pump or suction air chamber, and not be pocketed in some high part of the suction pipe. A slope of 1 per cent. will be found very satisfactory.

"Be sure that your suction piping is absolutely tight, for a very small air leak will cause a pump to work improperly. The suction pipe should be tested with about 20 lb. water pressure after it has been laid and before it is covered. If the test shows up a leak, fix it; it is not good enough.

"Keep the end of your suction pipe well under water. It should never have less than 3 ft. above it and 6 or 8 ft. will be much better.

"If two or more pumps draw from the same suction pipe, or if water comes to the pump under a head, a gate valve should be placed on the suction pipe of each pump, to enable you to open up any one pump cylinder for repairs or examination without interfering with the operation of the other pumps. We recommend on larger sizes where practicable and not too costly, that each pump have a separate individual suction line entirely independent of the suction line of any other pump.

"A suction air chamber will be found desirable in all cases, and indispensable in cases where the sum of the three items referred to in a previous paragraph (the third) exceeds 10 ft. or when the suction pipe is long.

"A foot valve[1] is desirable in all cases (except when suction water comes to the pump under a head) and indispensable when the suction lift exceeds 10 ft. By its use the pump and suction pipe are kept primed when the pump is shut down, and it permits of easily priming the pump and suction pipe if purposely emptied, thus enabling the pump to be easily started at any time.

"In all cases where the water contains sticks, weeds, rags, or other rubbish a strainer should be used on the suction pipe, to prevent them from getting into the pump and clogging valves and passages. If a foot valve is used, a strainer placed outside the foot valve is best; but if no foot valve is used, a box strainer placed near the pump and so designed that by removing the strainer cover all accumulations can be removed, will be found most desirable. Keep the strainer clear from accumulation of rubbish.

"When a foot valve is used, a drain valve should be placed near the surface of the water, to enable the suction pipe to be drained when desired.

"A relief valve, set to blow at about 20 lb. pressure, should also be placed on the suction pipe near the pump, to prevent the delivery pressure, if over 50 lb., from accumulating in the suction chamber of the pump or the suction pipe. This does not cost much and may sometimes save you the cost of replacing a broken pump cylinder or foot valve, due to carelessness.

"A check valve on the discharge pipe will be found very convenient. A gate valve should be placed on the discharge pipe outside the check valve.

[1] A foot-valve on a suction pipe for sewage is objectionable because of the great danger of its clogging with waste, cloth and other substances. It is the authors' opinion that such valves should be omitted whenever possible, thus eliminating the care necessary to keep them in order and reducing the friction while running as well as the danger of trouble. These comments also apply to strainers on the suction pipes.

"A priming pipe should always be connected from the discharge pipe, outside the gate, to the suction pipe, if a foot valve is used. This will enable the pump cylinders and suction pipe to be primed, if empty, before starting. If you have no suitable relief valve on the suction pipe, be very careful, in priming with this pipe, that you do not let delivery pressure accumulate in the suction pipe. This will be prevented by having the starting waste valve open before you start to open the priming pipe valve. This should always be open before starting your pump (whether you have a foot valve or not) as by this means the pump is enabled to discharge the air from the pump cylinders and suction pipe through this starting valve against a light pressure. As soon as water is discharged through the starting valve, shut it and open your steam throttle valve, and the pump will then discharge through the discharge main, opening the check valve automatically. If you have a foot valve or a gate on your suction pipe, and no relief valve, be careful to open the starting valve at the instant you shut the pump down and leave it open until after you have started again, as by so doing you prevent the possibility of pressure accumulating in the suction. The pet cock in the force chamber of small size pumps is intended to be used in the same manner as the starting valve above referred to.

"Do not pack the stuffing boxes too tightly, and do not let the packing stay in until it gets hard and cuts the piston rods or plungers. Renew it sufficiently often to keep it soft and pliable. If the pump runs badly, make sure that the pump valves, packed pistons or plungers, and suction and discharge connections are all right before examining the steam end."

CENTRIFUGAL PUMPS

There is some confusion in the use of names for different types of centrifugal pumps, and to settle the matter, the authors have obtained the following definitions from George de Laval, general manager of Henry R. Worthington, whose book on "Centrifugal Pumping Machinery" contains the only explanation in English of the methods actually used in designing this class of machinery for the highest practicable efficiency:

"Centrifugal pumps comprise all those pumps where the water is given a high rotary motion by an impeller, which velocity is then converted into head, forcing the water up to a certain height."

"Volute pumps are centrifugal pumps where the water is free to leave the impeller in any direction and is taken up by a gradually enlarging channel surrounding the impeller, by which gradual enlargement the exit velocity is changed into head. Here the conversion takes place in a very ineffective manner, hence the low efficiency."

"Turbine pumps are centrifugal pumps in which the water as it leaves the impeller is taken up by properly designed channels which are gradually enlarged and thereby reduce the velocity to such an extent that the losses in the surrounding channel are greatly reduced."

"Screw pumps do not properly belong to the type of centrifugal pumps ac-

they do not depend upon centrifugal force. They consist of a shaft provided with a vane forming a complete screw thread. The water travels in an axial direction, being propelled by the spiral vane."

"Propellor pumps are screw pumps in which only parts of the screw are utilised. These parts are arranged around a hub and form a screw of multiple pitch, the rotor being similar to a ship's propeller."

"Centrifugal-screw pumps have helical vanes developed on a conical surface. The pitch of the helix is constant for all radial distances from the axis. The water receives an axial or helical movement until it strikes the cone of the impeller, when it comes under the influence of centrifugal force."

Until comparatively recently little attention was paid to refinements in the design of centrifugal pumps in America, although they were used extensively. The greatest interest in their improvement was shown on the Pacific Coast down to about 1900, when the good results obtained with such apparatus in Europe led to a quite general interest in the betterment of its design. Previously American centrifugal pumps had been strong and durable, rather than efficient, but the attention paid to them since 1900 has resulted in an improvement in efficiency. Unfortunately this was not all that was needed, however, for the proper design of the power end of the equipment is as important as that of the pump, particularly when electric motors furnish the power. It was a rather surprising condition in the electrical industry for some years that the peculiar requirements of centrifugal pumps were overlooked in selecting motors for them, although the lack of efficiency in such combined units called attention repeatedly to the necessity of adapting the two ends of the plant to each other in a better way. Today, as Mr. de Laval has stated in his book already mentioned, "the designer of the pump must carefully consider the nature of his motor when laying out the characteristics of his impeller, and the electrical engineer should design his motor to suit the characteristics of the pump," where the plant must work against a variable head, in order to obtain the highest efficiency. This is generally recognized now and plants of this type are unquestionably more efficient than they were in 1900, although business competition, poor specifications and lack of tests to ascertain if guarantees have been met have the usual retarding influence on progress.

Special Features of Centrifugal Pumps.—The theory of the centrifugal pump as presented in most text-books in English is quite simple. It is assumed that the particles of water moving outward between the vanes of a revolving impeller are given a uniformly increasing linear velocity, so that all particles at the same distance from the center of the impeller have the same velocity. Then the reverse of the usual theory for a turbine will give an analysis of the pump, or the flow between the vanes of the impeller may be considered as the flow through pipes under pressure, but subject to change in the internal hydrostatic pressure due to

centrifugal force. Both methods result in the same equations, and both are seriously at fault, for practical purposes, because all particles of water at the same distance from the center do not have the same velocity. The subject is treated in considerable detail in Prof. L. M. Hoskins' "Text-Book on Hydraulics," but after reading his explanation of the general principles of centrifugal pump action, the practical application of them in de Laval's "Centrifugal Pumping Machinery" will show what a great difference exists between theory and practice and some of the reasons for it.

The performance of a centrifugal pump is shown by characteristic curves, Fig. 302, which reveal two important properties of such apparatus. The first is the impossibility of producing a greater pressure at any speed than that shown by the curve. Hence, if the discharge pipe should be

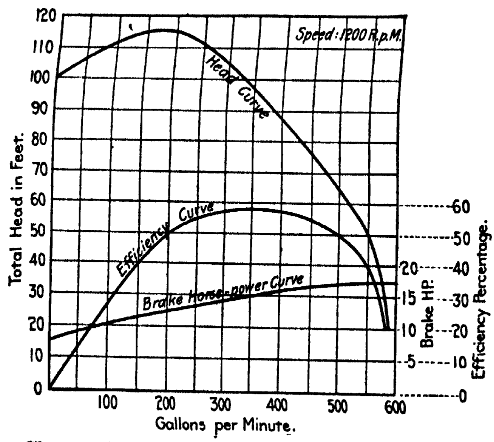


FIG. 302.—Characteristic curves of centrifugal pumps for fixed overload conditions (de Laval).

suddenly closed, there is no danger of rupturing it, as would be the case with a reciprocating pump. The second peculiar property of the pump is that at a given speed and for heads between certain limits there are two rates of discharge. At first sight this might be considered an indication of uncertainty of operation, which would be the case were the pump discharging into a large tank in the immediate vicinity, so that there were practically no friction head for the pump to operate against. In this case the pump having the characteristics shown in Fig. 302 would begin to discharge when the head of 100 ft. was reached, and would continue discharging until about 115 ft. head was reached, when the discharge would cease and the pump could not be made to deliver water.

again until the head dropped to 100 ft. Practically, this trouble is prevented by means of a gate on the discharge pipe. By partly closing this gate the head can be raised from 100 to 115 ft.; the pipe friction plays some part, also, in the regulation of the flow. If the friction head were increased by further throttling the discharge would be cut down, for capacity and friction head are related in an inverse ratio: if the friction head were reduced after it reached 115 ft. the discharge would be increased.

Another special property of the centrifugal pump must be kept in mind. If the head were suddenly reduced by a break in the discharge pipe or some equivalent cause, the pump would discharge more and more water until it reached its capacity for such a head. This would throw a greater load on the motor, however, and might injure it if the

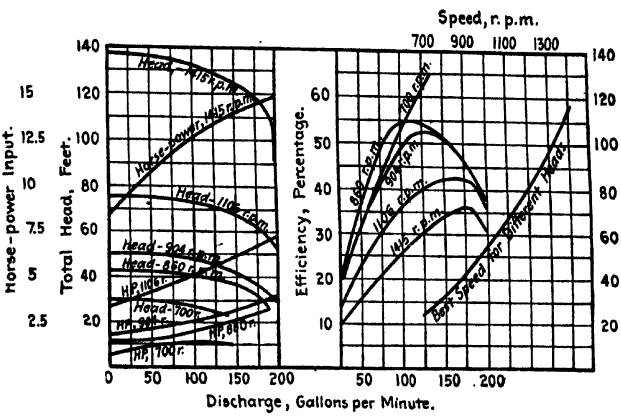


FIG. 303.—Performance of a 2-inch centrifugal pump under test.

impeller of the pump were not designed so that not more than a certain percentage of overload could be imposed in any case. Mr. deLaval says that this should not exceed 25 per cent.; it is sometimes considerably more than this.[1] The ordinary good design gives a capacity of 70 to 125 per cent. of that at maximum efficiency, with a 25 per cent. variation of head and about 5 per cent. variation of efficiency. A pump can be so

[1] In the Memphis sewage pumping station there are two horizontal centrifugals, one a 24- and the other a 20-in., on the engine-room floor and an 8-in. submerged centrifugal. All are driven by three-phase induction motors, the first by a 175 h.p., the second by a 150 h.p., and the third by a 50 h.p. The first two have a rated capacity 30 per cent. in excess of that required to drive the pumps under normal conditions. "The great increase in the quantity of water delivered by a centrifugal pump as its suction lift is reduced, resulting in a corresponding increase in the power required to drive the pump, led to the adoption of the excessive size of motors, so that the capacity of the station can be greatly increased as the water rises in the storm-water pump well." (*Eng. Record*, April 21, 1906.)

designed that the power curve will be nearly constant, according to de Laval, while the efficiency is maintained.

The characteristic curves of Fig. 302 were taken from a pump at constant speed, the usual basis for designing centrifugal pumping plants. A good centrifugal pump can be operated at a considerable range of speeds,[1] however, without any great loss in efficiency. This is shown in Fig. 303, which gives the curves obtained in a test of a 2-in. horizontal single-stage stock pump with an open impeller, tested at the New Mexico Agricultural Experiment Station by B. P. Fleming and J. B. Stoneking. The efficiency in this test was figured on a total head comprising the lift from the suction elevation to the discharge elevation plus the friction head plus the velocity head gained between the suction and discharge pipes. If the velocity head in the discharge pipe were less than in the suction pipe, the difference would have to be subtracted in ascertaining the total head.

This pump was supplied with an impeller in which the angle between each vane and the tangent to the circumference of the impeller was 30 deg. In order to ascertain what effect this angle had on the performance of the pump, three impellers were made having angles of 0, 60 and 90 deg., respectively, but none did so well as the stock impeller.

The impeller of a centrifugal pump is either open, when there are no plates on the sides, or enclosed, when each side has a plate and the water has no chance to touch the side walls of the pump casing during the entire time it is within the impeller. The enclosed type reduces the internal friction of operation somewhat, and is employed where economical operation is desired; it has been used for pumping sewage, and the larger sizes may be as well adapted for such work as pumps with open impellers. The latter are usually recommended for small sewage pumping plants with low to moderate lifts and Tables 167 and 168 give the data necessary to select the size best adapted for small stations. The former table was supplied by Henry R. Worthington in March, 1914, and relates to that firm's Class C pumps. The second table was supplied in April, 1914, by the Alberger Pump & Condenser Co. The tables of centrifugal pump capacities, speeds and power requirements printed prior to 1914 should not be used by the engineer without being checked, as material changes in them are needed. The reader is particularly cautioned against applying tabular data relating to specially designed pumps with enclosed impellers to stock pumps with open impellers, and also against confusing single and double suction pumps, and turbine and volute pumps. Finally it should be kept in mind that liberal water passages are needed in pumping sewage, and if the quantity to be handled is small, it may not be advisable to use the more efficient

[1] But electric motors of some types are incapable of much speed variation, so an electrically operated unit may be a very inflexible one, in spite of the capabilities of its water end.

TABLE 167.—RANGE IN CAPACITY, SPEED AND POWER REQUIREMENTS OF CLASS C WORTHINGTON SINGLE-SUCTION VOLUTE PUMPS WITH OPEN IMPELLERS FOR PUMPING SEWAGE AND WATER CONTAINING CONSIDERABLE GRITTY MATERIAL

(G, gallons per minute; HP, horse power required; R, revolutions per minute)

Head	10 ft.			15 ft.			20 ft.			25 ft.			30 ft.			35 ft.			40 ft.			50 ft.			60 ft.		
Size, in.	G	HP	R	G	HP	R	G	HP	R	G	HP	R	G	HP	R	G	HP	R	G	HP	R	G	HP	R	G	HP	R
1	14	0.15	1125	17	0.25	1378	20	0 36	1591	22	0.49	1780	24	0.62	1947	26	0.77	2107	28	0.94	2251	31	1.32	2514	34	1.73	2756
1	18	0.19	1177	22	0.32	1442	26	0.47	1663	29	0.63	1861	32	0.82	2041	34	1 03	2200	37	1.26	2354	41	1.77	2629	45	2.31	2882
1¼	21	0.22	990	26	0.37	1210	30	0.53	1397	34	0.72	1562	37	0.94	1711	40	1.18	1848	43	1.4	1976	48	2.0	2211	53	2.6	2420
1¼	28	0.28	1062	35	0.48	1298	40	0.70	1500	45	0.96	1678	49	1.26	1837	53	1.58	1986	56	1.9	2123	63	2.7	2371	69	3.6	2596
1½	42	0.4	836	52	0.7	1023	59	1.0	1181	66	1.3	1321	73	1.8	1450	78	2.2	1562	84	2.7	1672	94	3.8	1870	102	5.0	2046
1½	60	0.6	937	73	1.0	1147	84	1.4	1326	94	2.0	1481	103	2.7	1621	111	3.3	1753	119	4.0	1871	133	5.7	2094	146	7.5	2294
2	70	0.6	717	86	1.0	880	99	1.4	1015	111	1.9	1133	121	2.4	1243	131	3.0	1342	140	3.7	1434	157	5.2	1604	171	6.8	1758
2	99	0.8	791	121	1.4	968	140	2.0	1118	156	2.8	1251	171	3.6	1370	185	4.5	1477	198	5.5	1582	221	7.8	1768	242	10.2	1936
2½	94	0.8	578	115	1.2	708	133	1 8	817	149	2.3	914	163	3.0	1001	176	3.6	1082	188	4.4	1155	210	5.9	1294	230	7.7	1417
2½	122	1.0	618	150	1.6	758	173	2.3	875	193	3.1	978	212	3.9	1071	229	4.8	1155	244	5.8	1238	273	8.1	1384	299	10.6	1514
3	136	1.3	546	167	2.0	668	192	2.7	771	215	3.6	862	235	4.5	945	255	5 5	1021	272	6.5	1090	304	9.0	1221	333	11.3	1338
3	184	1.6	575	225	2.5	704	259	3.5	814	290	4.8	909	318	6.0	996	343	7.3	1076	367	8.7	1150	410	11.9	1285	450	15.5	1408
4	230	1.9	472	281	2.9	577	325	4.0	666	363	5.3	746	398	6.8	817	430	8.3	882	460	9.9	944	514	13.3	1055	563	17.3	1155
4	273	2.1	495	334	3.3	606	386	4.8	700	431	6.3	782	472	8.0	858	510	9.8	925	545	11.7	990	610	15.9	1104	669	20.6	1210
5	415	2.7	409	509	4.3	500	587	6.0	578	656	8.0	648	719	10.0	710	776	12.4	765	830	14.8	818	928	19.8	915	1016	25.7	1002
5	510	3.2	433	625	5.2	531	721	7.4	613	807	9.7	686	884	12.4	751	955	15.0	811	1020	18.1	867	1140	24.4	970	1250	32.0	1062
6	604	3.4	354	740	5.3	435	855	7.6	502	955	10.0	561	1047	12.6	614	1130	15.4	663	1208	18.3	710	1350	24.8	792	1480	32.0	869
6	786	4.3	385	965	6.9	470	1113	9.9	545	1245	13.2	607	1365	16.7	666	1470	20.5	721	1575	24.6	770	1760	33.4	858	1930	44.1	941
8	1094	6.2	305	1340	9.7	373	1550	13.6	431	1730	18.1	482	1898	22.8	528	2045	28.0	570	2190	33.3	609	2450	45.0	682	2680	58.0	746
8	1281	7.0	322	1569	11.0	395	1811	15.8	455	2025	21.2	509	2220	26.6	558	2395	32.7	603	2560	39.2	644	2865	53.4	721	3135	69.1	789
10	1715	9.6	267	2100	15.1	328	2425	21.3	378	2710	28.5	424	2970	35.7	464	3210	43.7	501	3430	52.3	536	3835	70.5	600	4200	91.0	657
10	2290	12.4	300	2808	20.6	369	3240	28.7	426	3625	38.2	475	3970	48.9	521	4285	59.3	562	4590	71.3	602	5130	97.5	672	5610	128.0	737
12	2325	13.2	224	2845	20.8	275	3285	29.1	317	3675	38.7	354	4025	49.0	388	4350	59.6	419	4650	71.6	449	5200	95.7	500	5700	123.0	550
12	2940	16.7	240	3600	26.2	295	4155	37.1	339	4650	49.5	380	5100	63.0	417	5500	76.8	451	5890	92.3	482	6580	125.0	539	7200	164.0	589

Note.—This table shows that in some cases pumps of different sizes may be used to do the same work; the choice enables the pumps with the best speed for the conditions to be selected, such as a small high-speed pump for belt drive or a larger slow-speed pump for direct connection to an engine. The slow-speed combinations of pump and engine are recommended for continuous service. The impellers are designed to limit the overload on the motor to 20 per cent.

TABLE 168.—CAPACITIES IN GALLONS PER MINUTE, SPEEDS IN REVOLUTIONS PER MINUTE, AND BRAKE HORSE POWER REQUIRED FOR ALBERGER STANDARD VOLUTE PUMPS WITH CLOSED IMPELLERS

Size, in.	Capacity in G.P.M.	Max. size, sq. mesh for strainer	Head	10 ft.	15 ft.	20 ft.	25 ft.	30 ft.	40 ft.	50 ft.	60 ft.	70 ft.
1½	65	¼ in.	Speed	735–955	895–1,155	1,035–1,345	1,160–1,510	1,210–1,510	1,330–1,730	1,490–1,930	1,630–2,120	1,760–2,290
			Power	0.5	0.7	0.9	1.1	1 2	1.6	2.0	2.4	2.8
2	100	¼	Speed	660–860	805–1,050	930–1,210	1,040–1,350	1,090–1,350	1,200–1,560	1,340–1,745	1,465–1,905	1,585–2,060
			Power	0.6	0.9	1.2	1.5	1.7	2.2	2.7	3.2	3.8
2½	150	⅜	Speed	470–615	580–755	665–865	745–970	780–970	855–1,110	955–1,240	1,050–1,365	1,130–1,470
			Power	0.9	1.3	1.7	2 0	2.3	2.9	3.6	4.3	5.0
3	250	½	Speed	550–715	675–880	780–1,015	870–1,130	910–1,130	1,000–1,300	1,120–1,460	1,225–1,595	1,325–1,720
			Power	1.3	2.0	2.5	3.0	3.5	4.6	5.7	6.8	7.9
4	450	⅝	Speed	470–615	580–755	665–865	745–970	780–970	855–1,110	955–1,240	1,050–1,365	1,130–1,470
			Power	2.3	3.4	4.2	5 2	6.1	7.9	9.7	11.5	13.3
5	700	⅝	Speed	415–540	505–660	585–760	650–845	680–845	750–975	835–1,085	915–1,190	990–1,285
			Power	3.4	5.0	6.3	7.5	8.7	11.4	14.2	17.0	19.8
6	1,000	¾	Speed	390–510	475–620	550–715	610–795	640–795	705–915	790–1,025	860–1,120	930–1,210
			Power	4.9	7.1	9.4	11.7	13.0	16.8	19.4	23.5	27.0
8	1,800	⅞	Speed	350–455	430–560	495–645	550–715	575–715	635–825	710–920	775–1,010	840–1,090
			Power	7.9	11.4	15.0	17.5	20.4	27.2	34.0	40.8	47.5
10	2,800	1	Speed	300–390	370–485	425–550	475–615	495–615	545–710	610–790	670–870	720–935
			Power	11.8	17.0	22.0	27.2	32.5	41.3	51.3	61.3	71.3
12	4,000	1¼	Speed	255–335	315–410	360–470	405–525	420–525	465–600	520–675	570–735	615–795
			Power	16.9	24.5	31.6	38.4	44.5	57.6	72.0	86.5	101.0
14	5,500	1½	Speed	230–300	280–365	320–415	360–465	375–465	415–535	460–600	505–655	545–710
			Power	23.2	33.6	43.5	52.6	61 4	79.5	99.4	119.0	139.0
16	7,500	1¾	Speed	210–270	255–330	295–380	325–425	340–425	375–490	420–545	460–595	495–645
			Power	31.6	46.0	59.3	72.0	84.3	109.0	136.0	163.0	190.0
18	9,500	2	Speed	185–240	230–295	265–340	295–380	305–380	340–440	375–490	415–535	445–580
			Power	40.0	58.0	75.0	91.0	106.0	138.0	173.0	207.0	242.0
20	12,000	2	Speed	170–220	205–270	240–310	265–345	280–345	305–395	340–445	375–485	405–525
			Power	50.6	73.5	94.5	115.0	135.0	173.0	216.0	260.0	303.0
24	18,000	2	Speed	140–185	175–225	200–260	220–290	230–285	255–330	285–370	310–405	335–435
			Power	76.0	110.0	142.00	172.0	202.0	260.0	325.0	388.0	455.0

Note.—The brake horse power in each case is based on water. For heavier liquids the power must be increased in proportion to the specific gravity. For the larger sizes, with favorable conditions, the efficiencies on which the above powers are based can be increased considerably. In selecting motors

types owing to their greater liability to interruption when running on a liquid containing many kinds of solids.

The screw pump is used where large volumes of water have to be moved against very low heads. In sewerage work, they have been employed mainly in producing a proper flow through large conduits supplying water for flushing rivers, as at Milwaukee, or tidal inlets, as at Brooklyn. The peculiarity of the pump is that a certain amount of shock is inevitable, and consequently the screw type can never be as efficient as a well-designed centrifugal pump. The latter, however, cannot be built to operate satisfactorily for some of the conditions for which the screw pump is well adapted.

Efficiency of Centrifugal Pumps.—The nominal efficiency of a centrifugal pump depends upon the total head, which has never been defined by any authoritative body, and at present there are three definitions of the term among engineers. The first is the algebraic sum of the discharge head and suction head, the second adds the velocity head in the discharge pipe to the first quantity, and the third adds the difference between the velocity heads in the suction and discharge pipes to the first quantity. The third meaning of the term is the accurate one, theoretically, for it represents the actual head pumped against. The pump must also overcome various resistances due to the churning of the water and other sources of water friction, the skin friction between the water and the impeller and chamber walls and other things, of all of which very little definite knowledge has been acquired yet. In spite of this lack of information there has been such great improvement in design that even small centrifugal pumps can now be had under a guarantee to show higher efficiencies than the 40 per cent. which was about the best obtainable from an ordinary volute pump in 1895.

According to George de Laval, 55 to 65 per cent. efficiency should be obtained with the most efficient type of pumps delivering 75 to 250 gal. per minute, 70 per cent. with those of 250 to 900 gal., 70 to 73 per cent. with those of 900 to 3000 gal., 73 to 75 per cent. with those of 3000 to 6000 gal., and 75 to 78 per cent. with those of 6000 to 10,000 gal. With pumps of capacities over 10,000 gal., 75 to 85 per cent. efficiency is obtainable, he states. Side entrance or single suction pumps give slightly less efficiency. The late William O. Webber expressed this effect of size on efficiency in *Engineering News*, Jan. 10, 1907, as follows: A 2-in. pump giving an efficiency of 38 per cent., a 3-in. pump giving 45 per cent., a 4-in. pump giving 52 per cent., a 5-in. pump giving 60 per cent., a 6-in. pump giving 64 per cent., are proportionally as good as an 18-in. pump with 77 per cent. efficiency and a 32-in. pump with 80 per cent.

The efficiency of small centrifugal pumping plants as actually operated in Southern California was tested in 1906 by J. N. LeConte and C. E. Tait for the U. S. Office of Experiment Stations, which published the

TABLE 169.—TESTS OF STEAM-DRIVEN CENTRIFUGAL PUMPING PLANTS (LE CONTE AND TAIT)

No.	Engine	Pump, centrifugal	Discharge sec.-ft.	Indic. h.p.	Water h.p.	Suction, ft.	Discharge, ft.	Runs averaged
1	10" × 6" simple non-con. 75 h. p. nom.	Vert. comp. 6" suc., 7" dis., 1003 r.p.m.	0.578	31.8	10.7	32.0	132.0	7
2	9" × 12", simple, non-con. 35 h.p. nom.	Vert. comp., 8" dis., 700 r.p.m.	1.37	31.7	14.7	0.0	94.3	7
3	10½" × 30", simple condens. Corliss 118 r.p.m.	Vert. comp., 9" suc., 10½" dis., 803 r.p.m.	0.89	107.8	41.3	17.8	195.6	11
4	8", 12" × 22" × 16" trip. conden., 140 r.p.m.	Hor. single two 20" suc., 30" dis., 140 r.p.m.	54.77	139.0	83.7	13.54	−0.06	9
5	8½", 12¾" × 22" × 16", trip. conden., 150 r.p.m.	Hor. single two 20" suc., 30" dis., 150 r.p.m.	52.02	134.0	78.9	13.54	−0.06	7
6	16¼" × 28" × 20" comp. conden., 151 r.p.m.	Hor. single, 44" suc , 44" dis., 151 r.p.m.	95.4	239.0	106.4	15.0	−5.0	15
7	16" × 26" × 18", comp. cond., 164 r.p.m.	Hor. single, 44" suc., 44" dis., 151 r.p.m.	87.4	216.0	96.4	15.0	−5.0	13

TABLE 170.—TESTS OF CENTRIFUGAL PUMPING PLANTS DRIVEN BY INTERNAL COMBUSTION ENGINES (LE CONTE AND TAIT)

No.	Engine	Pump, centrifugal	Suction, ft.	Head, ft.	Discharge sec.-ft.	Ind. h.p.	Water h.p.	Runs averaged	Fuel
1	6" × 12", 240 r.p.m., 11 h.p. nom.	4" single vert.	0	44.4	0.328	5.64	1.65	15	Distillate
2[1]	8 5/16" × 14", 12 h.p. nominal.	4" single vert.	20.5	41.4	0.671	16.3	4.70	9	Distillate
3	16" × 18", 180 r.p.m. 50 h.p. nom.	10" single vert.	0	11.31	5.94	38.0	7.60	8	Distillate
4	11½" × 18", 200 r.p.m. 32 h.p. nom.	5" comp. vert.	8.3	88.3	0.829	26.5	9.05	12	Distillate
5	9½" × 20", 260 r.p.m. 30 h.p. nom.	8" single vert.	18.6	49.3	1.36	35.2	10.4	13	Distillate
6	10" × 18", 30 h.p. nominal.	6" single vert.	21.0	41.0	1.51	29.3	10.6	7	Distillate
7	10" × 18", 185 r.p.m. 30 h.p.nom.	7" comp. vert.	20.5	73.0	1.09	26.7	11.4	16	Distillate
8	11" × 20", 180 r.p.m., 35 h.p. nom.	7" comp. vert.	23.0	67.4	1.33	40.8	13.6	8	Distillate
9	16½" × 22", 180 r.p.m. 60 h.p. nom.	12" sing. horis.	6.5	48.5	4.96	63.1	30.9	13	Distillate

[1] Same pump and setting used also in Test 1, Table 171, electrically driven plants.

results in its Bulletin 181. Tables 169, 170 and 171 summarise the leading results and indicate the service the plants were giving without preliminary tuning up for tests.

The internal combustion engines were using, as a rule, engine distillates of 35° to 48° Beaumé. The physical condition of the plants varied widely. A fair average of the conditions indicated a probable annual overhead charge of 12 to 15 per cent. for depreciation, 6 per cent. for interest and 1 per cent. for taxes and insurance. The amount of distillate used per indicated horse-power-hour varied rather regularly from 0.154 gal. for the smallest plant to 0.1 gal. for the largest. The amount used per water horse-power-hour varied more widely, for it depended not only on the engine but also on the general efficiency of the entire plant; a fair average was from 0.5 gal. for the smallest plant to 0.2 gal. for the largest.

The electrically operated plants showed a higher plant efficiency than those driven by internal combustion motors. Le Conte and Tait concluded from the tests in Table 171, and others where different types of pumps were used, that electrically operated plants of a capacity of 5 water-horse-power should have 40 per cent. efficiency, and the efficiency should rise with the capacity to 55 per cent. for a plant of 40 water-horse-power.

TABLE 171.—TESTS OF CENTRIFUGAL PUMPING PLANTS DRIVEN BY ELECTRIC MOTORS (LE CONTE AND TAIT)

No.	Motor	Pump, centrifugal	Suction, ft.	Head, ft.	Discharge, sec.-ft.	Kilowatts	Water h.p.	Runs averaged
1	15 h.p. induc. 60 c., 3-ph., 220-v.	Vert. single, 4″ suc., 6″ dis. 990 r.p.m.	20.5	41.4	0.714	10.8	5.01	9
2	20 h.p. induc. 50-cyc., 3-ph. 440-v.	Vert. single, 8″ suc., 10″ dis. 446 r.p.m.	19.5	3.0	2.21	9.4	5.62	7
3	30 h.p. induc. 60-cyc., 3-ph. 440-v.	Vert. single 5″ and 6″ suc., 7½″ dis., 653 r.p.m.	0	60.3	1.26	16.4	8.57	11
4	50 h.p. induc. 7200 alt., 3-ph., 2000-v.	Vert. comp., no. suc., 14″ dis., 906 r.p.m.	0	28.4	2.97	41.2	9.45	9
5	40 h.p. induc., 60-cyc., 3-ph., 440-v.	Vert. comp. 8″ dis., 712 r.p.m.	0	94.3	1.38	23.8	14.7	7
6	30 h.p. induc., 60-cyc., 3-ph., 550-v.	Vert. comp. 7″ suc., 8″ dis., 723 r.p.m.	25.1	97.3	1.10	23.6	15.3	9
7	100 h.p. induc., 7200 alt., 3-ph., 400-v.	Comp., 6″ suc., 10″ dis., 790 r.p.m.	0	189.0	1.82	53.1	39.0	13

The steam-driven plants all burned crude oil as a fuel, and their efficiency did not differ definitely from the efficiency for gasoline or electric plants. The smallest plants required about 2.5 gal. of crude oil per water horse-power-hour and the largest plants required about 0.5 gal.

The investigation convinced Le Conte and Tait that there was a lack of good design and maintenance about most of the internal-combustion plants visited which, could it be corrected so as to bring about at every plant the same efficiency found in the best plant, would have reduced the consumption of gasoline in 1905 from 90,000 to 63,000 gal. In most of the plants the annual fixed charges for interest, depreciation and taxes far exceeded the expense for gasoline, attendance and repairs.

Setting Centrifugal Pumps.—There is considerable difference of opinion regarding the best arrangement of the pump as respects its supply. Some engineers favor submerging it; but pump makers oppose this, mainly because an exposed pump receives better care than one which is submerged in sewage. Others place it as near as possible to the water level in the suction well. William O. Webber stated in *Engineering News*, Jan. 10, 1907, that suction lifts of 10 to 15 ft., with enlarged suction pipes and taper connections, would give better efficiencies than were obtainable with a submerged pump.

A type of centrifugal pump setting has been developed for low-head irrigation work which has certain advantages where it is applicable. The pump is at the highest point of the suction and discharge pipes, and with them forms a siphon. This arrangement has been adopted by George G. Earl for the eleven new pumps for the New Orleans drainage system. These are of the screw type (*Eng. News*, Jan. 15, 1914) each of 322,000,000 gal. daily capacity against a head of 5 to 10 ft.

There must be no vertical bends in the suction pipe where air can collect, and, as in all piping of this class, special pains must be taken to make the joints air-tight. At the sea level it does not pay to try to use a higher suction lift than 25 ft.; the greatest suction lifts at elevations of 2640 and 5280 ft. are 20 and 16 ft., respectively. The suction pipe is usually one size larger than the discharge pipe.

The discharge piping should be as straight as possible, and it is sometimes considered advisable to bolt an increaser to the outlet of the pump so as to make the discharge pipe as large as the suction pipe.

In starting a centrifugal pump after priming it, the valve of the discharge pipe should be closed until the impeller is running at its normal speed, when the valve should be opened slowly. In case the head should be reduced below that for which the pump was designed, the discharge pipe valve should be partly closed at once so as to throttle the discharge by creating an additional friction head and thus prevent overloading the motor.

Priming is the process of expelling air from a centrifugal pump before it is started, for if the impeller runs in air it cannot create enough vacuum to raise water to its level. If the pump is always submerged or receives its supply under a head, priming is not needed. If the pump handles hot water or any other liquid giving off a vapor, the supply of liquid must

reach the pump under some pressure, in order to prevent the collection of vapor in the chamber, which will stop the discharge.

If the suction pipe has a foot-valve to prevent backward flow, the simplest method of priming is to fill the pump and the suction pipe with water from a street pipe or other permanently reliable source, which can be admitted through a valve tapped into the top of the casing. In some cases it may be necessary to lift the water from the well into the casing of the pump by a steam injector. Where a foot valve is not used and the pump is not supplied with water under a head, a check valve may be placed in the suction pipe close to the pump and an injector may be tapped into the suction pipe just below the valve, with its discharge pipe tapped into the top of the pump casing. Another method of priming is by exhausting the air in the pump casing and suction pipe, which results in water being forced into them by atmospheric pressure. If a steam or water ejector or an exhaust pump is tapped into the top of the pump casing, the discharge pipe must always be closed while the ejector is drawing water up the suction pipe and into the casing.

Where the heads against which the pump works exceed about 30 ft., a check valve is usually placed in the discharge pipe near the pump, in order to protect the latter. The casing of the pump near the center of the sides is not strong and it is very difficult to brace it with ribs. If the pump were to stop running suddenly the sudden checking of the velocity of the water in the pump would cause a heavy pressure on these relatively weak portions of the pump, particularly if a foot valve were used. If a check valve is employed as suggested, a pipe can be tapped into the discharge pipe just above it, and water for priming can be obtained in this way. Centrifugal pumps as large as 12 in. can also be primed by means of "priming elbows" between the suction opening of the casing and the suction pipe. These elbows are provided with small hand-pumps which draw water through the main suction pipe and deliver it to the pump, where it is retained by a clack valve in the elbow. Various other modifications of these methods have been used.

An elaborate system of priming was installed at the pumping station at Salem, Mass. Here there are four horizontal centrifugal pumps of 6,667,000 gal. capacity each, electrically driven. The priming is done by two Knowles 4 × 4-in. dry-air vacuum pumps driven by a General Electric motor of 5½ h.p. The installation is illustrated in *Engineering News*, May 28, 1908, and is so arranged that any one or any combination of the main pumps may be primed by the use of either or both of the priming pumps. One-inch pipes are run from the highest part of the pump chambers to the back of the switchboard of the station, where there are valves controlling each line, and the pipes are then joined and connected to the bottom of an air-tight chamber. Suction pipes from the priming pumps are connected to the top of the same chamber in

which is placed a balanced port valve connected with a copper float which controls the valve. This arrangement was designed to prevent the drawing of sewage into the vacuum pump. To each of the priming pipes between the valves and the main pump there is connected a combined pressure and vacuum gage, with a dial mounted on the wall near the board and on the side of the air-tight chamber there is placed a water gage. It is stated that this arrangement has proved satisfactory except for trouble from leaves and similar objects which enter the chamber; this has been remedied by placing a screen box in the pipe from the main pumps.

The foot valve[1] at the bottom of the suction pipe should have an area about 50 per cent. larger than that of the suction pipe. At the bottom it is often provided with a strainer with openings large enough to permit the passage of all objects which will not obstruct the passages in the impeller. This strainer should not be the main reliance to prevent sticks and other objects from entering the pump, but should be regarded as an additional precaution. If the foot-strainer is relied upon to do all the screening, it is likely to become clogged speedily where sewage is pumped; in fact, a foot valve and a strainer on a sewage pump are very objectionable and should only be used when absolutely necessary. The clacks in the valve should have their hinges on the outside of the valve-seat plate, so that when they are raised they will offer as little obstruction as possible to the passage of the sewage.

Centrifugal pumps must be held firmly in position and all shafting must be well supported to secure satisfactory operation. "A combined bedplate for the pump and motor should be leveled up by wedges, the pump and motor placed upon the facing strips and lined up so that the faces of the pump coupling are parallel, and the pump and motor run freely with and without coupling bolts in position. The bedplate should then be grouted into place so that it is absolutely rigid. After the foundation bolts have been permanently set the suction and discharge piping may be connected" (De Laval).

"The best bearing for the vertical shaft is an important element in the design. At Saratoga, where a bearing several inches in diameter, with alternate loose rings of brass and steel, submerged in oil, was employed, considerable trouble was encountered because of heating. At Hudson, a regular Reeve's propeller bearing with an oil-collecting pan and the oil lifted and circulated by centrifugal force, as is done in motor work, was used with entire success." (Frank A. Barbour.)

The Saratoga pumps were three in number with 6-in. discharge orifices, and were driven by 20-h.p. induction motors. The Hudson pumps were 5-in. driven by 15-h.p. induction motors. The former had a combined efficiency of about 55 per cent. and the latter of 42 per cent.

The shafts of vertical pumps should be steadied by bearings 6 to 10 ft.

[1] See footnote on page 661.

apart, vertically, for such pumps are somewhat more difficult to operate than those with horizontal shafts. The smaller the shaft, the closer should be the steady bearings. A slip coupling in the vertical shaft between the motor and pump may be desirable. If the vertical shaft is short, the thrust bearing supporting the shaft and impeller may be in the top of the pump frame, but if the shaft between the pump and motor is a long one, or the pump is submerged, an independent thrust bearing at the top of the shaft just under the motor is desirable. The bearings for the pump and motor are standardized by each manufacturer, but the purchaser should satisfy himself that they are ample for the hard service of sewage pumping.

Prime Movers.—If a centrifugal pump is driven by a motor, the latter should not be too small or it will operate under an overload much of the time; if it is too large, the cost of power will be needlessly high. The size must be based on a consideration of both the normal and maximum conditions. If the head varies, it is desirable to change the speed of the pump, and the motor must therefore permit speed regulation, or some such form of control as that used at Dallas and Lebanon by James H. Fuertes, described later in this chapter, must be adopted. The proper design of a combined electric unit calls for special knowledge and for ordinary sewage pumping installations the best equipment will probably be obtained when the working conditions are stated fully and manufacturers are left to furnish the machinery under guarantees as to its efficiency and capacity. In handling sewage, slow speed and low efficiency are not such drawbacks as questionable reliability.

"When direct current is available, it is advisable to use motors of the variable speed type, especially in cases where the head or capacity is subject to change. As standard induction motors run only at constant speed, it is necessary to vary the capacity of the pumps by throttling the discharge; when the capacity or head changes considerably, it is most economical to accomplish the work with two units, operating then in series or parallel as the service demands. The shunt-wound direct-current motor is usually employed for driving centrifugal pumps, but in cases where the voltage or load fluctuates considerably, better results can be obtained with the compound wound motor. This type is also recommended when the motor is automatically started. For alternating-current motors, the squirrel-cage type is most frequently selected. This type of motor, however, requires a high starting current, and should not be used when the power available is limited, as it causes a disturbance in the line. The slip-ring motor takes a very small excess current at starting, and is therefore recommended in such cases." (Henry R. Worthington.)

The utility of electrically-driven centrifugal pumps for small sewerage systems is shown by some figures in the 1911 and 1913 reports of Chief Engineer Dexter Brackett of the Metropolitan (Boston) water-works.

The pumping of the sewage of Clinton, Mass., in the former year was done by a steam-driven plunger pump and in the latter year by a 12-in. single-stage centrifugal pump driven by a 40-h.p. squirrel-cage motor. In 1911 an average of 829,000 gal. of sewage was pumped daily and in 1913 1,008,000 gal. The labor charge in 1911 was $1,715.34, fuel cost $1,104.88, and repairs and supplies $194.63, a total of $3,014.85. This gave 20.1 cents per 1,000,000 gal. as the total cost of pumping 1 ft. high. In 1913 the charge for labor was $1,342.51; current, at $5.30 per 1,000 kw.-hr., $603.82; coal for burning sludge and heating, $227.64; repairs and supplies, $321.30; total, $2,495.27, or 13.8 cents per 1,000,000 gal. 1 ft. high. This figure of 13.8 cents averages 18 per cent. less than the cost during the previous 13 years of operation.

Sewage pumps are usually of the volute type, as the heads are so low that the diffuser of the turbine type is not worth its cost. The single suction pumps have a casing of relatively large diameter, and are therefore preferable for low and moderate speed prime movers and for belt drives. Manufacturers do not usually advise their selection where the heads are more than 80 ft. The double suction volute pumps have much smaller casings than the single suction pumps of the same capacity and consequently can be run at high speed for which they are best adapted. When so operated they will work well against heads of 150 ft. and even more in well-designed and operated plants. They are frequently used with direct-connected steam turbines. Vertical double-suction volute pumps are used in the sewage pumping station at Havana, Cuba.

There are two distinct types of centrifugal installations driven by steam engines. The first uses a high-speed engine, with a maximum speed of about 800 r.p.m. and an average speed of about 600 r.p.m.; These are not high speeds for centrifugal pumps, however, and consequently steam-driven units frequently have larger inpellers than those run at the higher speeds which are regularly employed with direct-connected motors. For small capacities, a simple engine is used, while for larger capacities a compound engine is needed at times, in which case the pump is mounted between the high-pressure and low-pressure ends of the unit, on the same baseplate. Complete engine-driven units are supplied by the pump manufacturers in many sizes and capacities, but on account of the speed limitations they are not available for all purposes for which centrifugal pumps can be used. Where large amounts of sewage have to be handled, and the pumps run continuously for long periods, the engines are compound condensing, and sometimes triple expansion. The first American triple-expansion engines for sewage pumping were probably those in the pumping stations of the Boston Metropolitan sewerage district. The centrifugal pumps had vertical shafts with a crank at the top; the engine cylinders were horizontal, arranged radially about the pump shaft, to the crank of which each

apart, vertically, for such pumps are somewhat more difficult to operate than those with horizontal shafts. The smaller the shaft, the closer should be the steady bearings. A slip coupling in the vertical shaft between the motor and pump may be desirable. If the vertical shaft is short, the thrust bearing supporting the shaft and impeller may be in the top of the pump frame, but if the shaft between the pump and motor is a long one, or the pump is submerged, an independent thrust bearing at the top of the shaft just under the motor is desirable. The bearings for the pump and motor are standardized by each manufacturer, but the purchaser should satisfy himself that they are ample for the hard service of sewage pumping.

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(*Eng. News*, Dec. 2, 1909, and April 17, 1913), although never used for sewage, is a new apparatus which may prove useful when its performance in water-works service has been of sufficient duration to show what are its practical merits and drawbacks. An Adams sewage lift, such as is used in a number of English towns, was employed with satisfactory results in Salem, N. J., until it was abandoned in 1912 on account of the reconstruction of the sewerage system. Such sporadic installations of unusual apparatus for raising sewage are too rare to merit description here, and among special pumps the only type that now (1914) has an established position in sewerage work is the ejector, of which the Ellis, Shone, Priestman, Pacific and Ansonia apparatus may be mentioned as examples.

The general arrangement of an ejector plant may be explained by a brief description of an installation of Ellis apparatus made in Schenectady in 1907, under the direction of City Engineer L. B. Sebring. The purpose of the plant was to deliver house sewage of a low-lying district across a high ridge which would require very heavy trenching if a gravity sewer were installed. The machinery was placed in a 26 × 11 ft. concrete chamber below the street surface, and comprised four ejectors each of a capacity of 100 gal. per minute, operated by compressed air supplied through a storage tank by two electrically driven compressors with a combined capacity of 340 cu. ft. of free air per minute. The ejectors were connected to an 8-in. pipe header leading to a 10-in. inlet pipe.

As soon as an ejector was full of sewage a valve at its top was automatically tripped, admitting compressed air from the storage tank at a pressure of about 30 lb. per square inch, which discharged the sewage through an 8-in pipe leading to a gravity sewer about 1900 ft. away. The vertical lift was about 21 ft. As soon as the contents of the ejector were discharged, the compressed air was automatically cut off and the ejector was ready for service again. The ejectors operated in rotation, the interval between discharges being determined by the rate of flow of the sewage.

The motors and compressors were placed at one end of the chamber, about 4-1/2 ft. above the floor on which the ejectors rested. After each discharge the supply of air in the storage tank was automatically replenished. The tank was fitted with a pressure regulator, and when the pressure fell below a predetermined point the hand on the gage made an electric contact which caused compressed air to be admitted to a piston operating the starting rheostat of one of the motors. When the operation of the compressor had brought the pressure in the storage tank to the proper amount, the pointer on the regulator made a second contact and the motor was automatically stopped. Under ordinary conditions only one motor and compressor were required to supply

the tank, and the second compressing outfit was held in reserve. If the first compressor failed to operate, the second motor was put in operation automatically by the pressure regulator on the air tank, which had a second electric contact point on its dial set for a lower pressure than the one first mentioned. An alarm system was also installed in a neighboring fire department house, which rang if both motors failed to operate.

At Cambridge, Ohio, the sewage of a small suburban district is raised about 35 ft., not including the friction head in 500 ft. of 6-in cast-iron force main, by a Priestman ejector, supplied with compressed air by a compressor which is driven by a Backus water motor. Water for the motor is supplied free by the city. The plant with the building, but excluding the force main, cost $2631.

Apparatus of this general type is manufactured by a number of companies and is rather widely used, although comparatively few installations have been made on city sewers. The main field of such ejectors has been in connection with the drainage systems of large buildings having basements and cellars below the elevation of the street sewer, so that the sewage and liquid wastes from these parts of the structures must be pumped. The only type of ejectors which has been extensively employed in municipal work is the Shone. One of the first important plants of this sort in the United States was at Winona, Minn., and after it had been in service for some years a second plant of the same type was introduced. Another installation which attracted considerable attention when it was put in was made at Fairhaven, Mass. Of late years it has been overloaded at times. The clerk of the Board of Sewer Commissioners, Norman M. Paull, informed the authors in 1913 that the ejectors have operated very well considering local conditions. Two of the four stations where they are located are in particularly wet places and although the chambers are either of cast-iron segments calked with lead, or boiler plate, they are by no means watertight, and many times the ejectors are partly or entirely submerged. Six of the ejectors had been in use 17-1/2 years and two of them for 9-1/2 years when they were overhauled, and their condition was good. A feature of their operation which has to be considered in February and March is the formation of ice in the valves and pipes through which the air escapes.

At Far Rockaway, a leading seashore resort in the Borough of Queens, N. Y., there are three Shone ejector stations and two automatic electric stations. Each ejector station contains two 250-gal. units furnished with air at about 20 lb. pressure by three compressors in the main sewage pumping station of the place. The electrically operated stations are of much larger capacity.

The Shone ejector as before stated, is operated by compressed air. Its general appearance and method of operation are indicated in Fig. 304 and the accompanying description furnished by the makers:

"It consists essentially of a closed vessel furnished with sewage inlet and discharge connections of a diameter suitable to the size of the ejector and the amount of sewage to be pumped. Each of these connections is furnished with a check valve (*A* and *B*) opening in opposite directions with regard to the ejector. On the cover of the ejector is placed the automatic valve *E*, to which is connected the pressure pipe from the air compressing station. This valve controls the admission of air to an exhaust from the ejector.

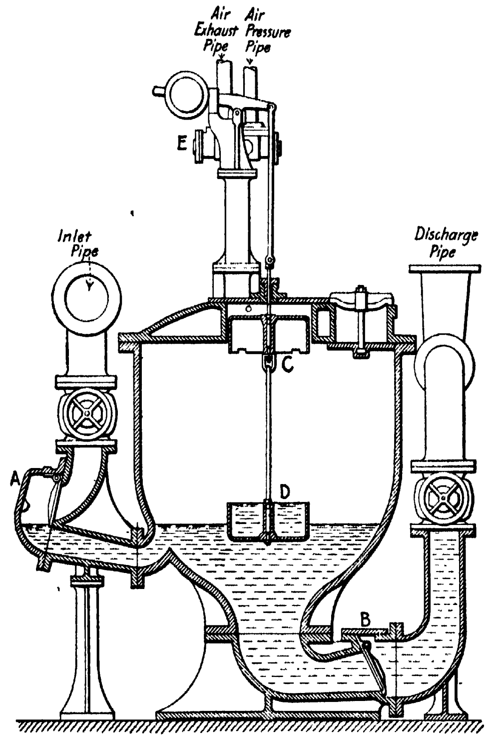

Fig. 304.—The Shone sewage ejector.

Inside the ejector are two cast-iron bells, *C* and *D*, linked to each other in reverse position, as shown, by a rod. A bronze rod to which the bell *C* is bolted passes through a stuffing box in the cover of the ejector and connects by means of links to a lever with a counterweight. The rising or falling of these bells operates the automatic valve *E* through a rocking shaft connecting it with the center of motion of the lever.

"As shown, the bells are in their lowest position (the extent of their movement being only about 1-1/2 in.), the compressed air is cut off from the

ejector and the inside of the ejector is open to the atmosphere through the automatic valve. The sewage therefore can flow from the sewers through the inlet valve *A* into the ejector, which it gradually fills until it reaches the underside of the bell *C*. The air at atmospheric pressure inside this bell is then enclosed, and the sewage continuing to rise around it, its buoyancy throws the system of counterweight and bells, etc., out of equilibrium. The bells consequently rise and the automatic valve is thrown over, thereby closing the connection between the inside of the ejector and the atmosphere, and opening the connection with the compressed air. The compressed air thus automatically admitted into the ejector presses on the surface of the sewage, driving the whole of the contents before it through the bell-mouthed opening at the bottom and through the discharge valve *B* into the iron sewage discharge main. The sewage can only escape from the ejector by the discharge pipe, as immediately the ejector is filled the inlet valve *A* falls on its seat and prevents the fluid returning in that direction.

"The sewage passes out of the ejector until its level falls to such a point that the weight of the sewage retained in the bell *D*, which is no longer supported, is sufficient to pull it down together with the upper bell and the parts to which it is connected, thereby reversing the automatic valve and returning it to its original position. The result of this action is first to cut off the supply of compressed air to the ejector, and then to allow the air within the ejector to exhaust down to atmospheric pressure. The discharge valve *B* then falls on its seat, retaining the liquid in the sewage discharge main; and the sewage flows through the inlet pipe into the ejector once more, and so the action goes on as long as there is sewage to flow and compressed air to drive."

The first Shone ejector installed at Worcester, Mass., was located at the sewage treatment plant, where it was used for lifting sludge, which flowed by gravity from sedimentation basins to storage basins, from which it was conveyed to the filter presses. This ejector has a capacity of 500 gal. per filling and is provided with supply and discharge pipes 12 in. in diameter. This apparatus was selected for this service because of its ability to handle successfully unscreened sludge, and has rarely been stopped by obstructions.

The Lake View installation, the third in the city, consists of a power house supplying compressed air to five Shone ejectors, which lift the sewage from a residential district having a population of about 1000, the flow amounting to about 20,000 gal. per day. This district is located on a side hill and is divided into three sections, low, intermediate and high level districts, each served by an ejector station. That serving the lowest raises the sewage about 50 ft. to the second station, which, in turn, raises this sewage together with that from its own tributary district to the third station, the intermediate lift being about 70 ft. The third station lifts the combined flow from the low and intermediate districts, together with the flow from the district tributary to the high level station to the summit, some 65 ft. above. The total lift of the

three stations is 189 ft. The general arrangement of these ejectors and the tributary sewer districts is shown by Fig. 305.

The power plant consists of two compressors, each driven by a 15 h.p. electric motor. The discharge pipes lead to a steel receiver from which the air passes through a wrought iron main to the several ejectors. The lowest ejector has a capacity of 150 gal. and the other four have capac-

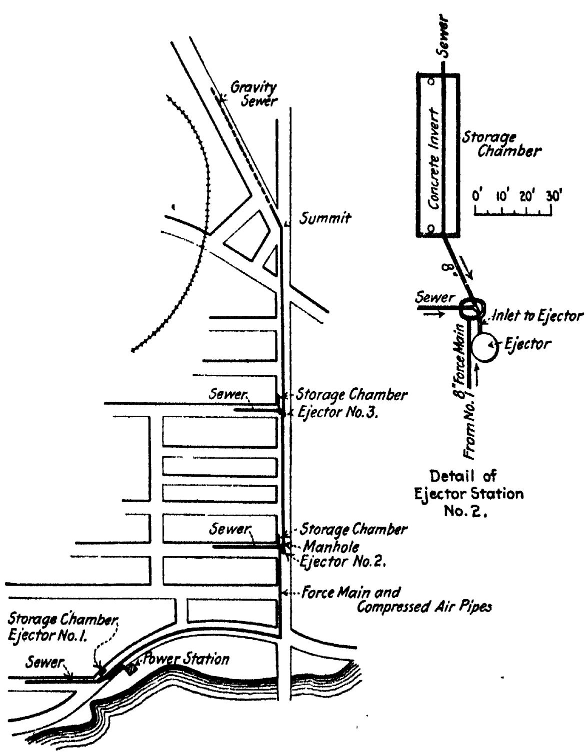

FIG. 305.—Shone sewerage system, Worcester.

ities of 100 gal. each. Number 1 ejector well is made large enough so that two ejectors can be acommodated when the flow of sewage becomes large enough to make additional machinery necessary. Close by each of the ejector wells and connected to it is an underground concrete storage tank, having a capacity of about 30,000 gal. These are necessary to provide storage for the sewage in case an ejector fails to

operate. The force mains consist of 2400 ft. of 8-in., and 2140 ft. of 10-in. cast-iron pipe.

The cost of this installation was approximately as follows: Ejectors, $2950; machinery, $1421; air and force mains, $8555; ejector wells and storage chambers, $5587; total, $18,513.

A test of this installation was made on March 20 and 21, 1906, at which time it was found that the efficiency was about 17 per cent., based upon the electric current delivered at the switchboard.

The cost of labor at that time, per million gallons raised 1 ft., was about $0.68.

While this system has the apparent advantages of being automatic and of not requiring that the sewage be screened, it is found in practice that considerable attention is required to keep the apparatus in good working order, particulary during the winter, when there is a tendency for the sliding valve to freeze. Little adjustment is necessary but the floats should be inspected and cleaned at frequent intervals and the apparatus should be kept oiled. At Worcester, it is the practice to have each ejector examined at least once each day. While it has not been found necessary to screen the sewage, thus avoiding the production of unpleasant conditions in the neighborhood, there has occasionally been some trouble due to sticks and other obstructions lodging under the valve *B*. When this happens the ejector is filled and emptied in quick succession, the sewage in the force main passing back through check valve *B* into the ejector. This, of course, results in the use of large quantities of air and if the valve is open so that the backflow is large, the air may be so drawn down that the station cannot maintain the necessary pressure and all of the ejectors in the system are thrown out of use.

PUMPING STATIONS

Pumping stations have been classified in a variety of ways, such as according to capacity or nature of prime movers, but there is nothing gained by such an artificial analysis. The authors have accordingly prepared brief descriptions of a number of stations, which illustrate the great variety of ways in which the problems due to poor foundations, variable capacity requirements, and different methods of obtaining power, have been solved. In some cases details have doubtless been employed which were due to local conditions and would not be selected for a standard design; in studying the various plans, particularly the type of pump drive, this influence of local conditions should not be overlooked.

Columbus, Ohio.—A sewage pumping plant built at Columbus, Ohio, from the plans of John H. Gregory (*Trans. Am. Soc. C. E.*, vol. lxvii, p. 282) is shown in Fig. 306. That engineer's description of it is as follows:

"The sewage is first admitted to a long chamber, serving as a sand-catcher, is screened to remove the coarser matters in suspension, and then passes into the suction well. The screening device consists of two cages, of steel-frame construction, holding removable sets of screens made up of 3/4-in. square bars, 1 in. apart in the clear. The cages are raised and lowered by hand by a movable screen lifter hung from a traveling hoist and runway just below the ceiling of the screen-room above. The substructure is of concrete, reinforced at various points. In the substructure of the engine room, in which are located the pumps and engines on account of the suction lift, the walls are lined with hard vitrified red pressed brick.

"The walls of the superstructure are of brick, faced with red pressed brick outside. In the engine room the walls are lined with light buff-speckled pressed brick, and in the screen room with hard red brick. The stone trimmings are all of Bedford limestone. The ceilings in both rooms are all of plaster on metal lath, fastened to the lower chords of the roof trusses. The roof is of 3-in. hollow terra-cotta tile and slate carried by steel trusses and intermediate framing.

"The pumping machinery is installed in duplicate. Each unit consists of a Columbus, horizontal, four-stroke-cycle gas engine connected by a Morse silent-running high-speed chain to a horizontal, single-stage Worthington volute pump with 12-in. suction and 10-in. discharge nozzles. The engine is capable of developing 90 h.p. when operating on natural gas having a thermal efficiency of about 1000 B.t.u. per cubic foot. When running together each unit has a rated capacity of 2,200,000 gal. per 24 hours against a head of 75 ft., and when running alone a maximum capacity of 2,900,000 gal. per 24 hours against a head of 63 ft. For starting the engines, the equipment includes a small motor-driven air compressor and air tank.

"The sewage is pumped through a 20-in. cast-iron force main to a point about 8180 ft. from the pumping station, where it is discharged into the upper end of the Mound St. sewer. The flow is measured by a 20-in. Venturi meter, the register, chart recorder and manometer being placed in the pumping station. The meter tube is of special construction, and between the tube and the register and manometer, oil seals are interposed to keep the sewage out of the latter."

Newton, Mass.—A pumping station built for temporary service at Newton, Mass., from the plans of the late Irving T. Farnham, illustrates a type of plant where the water end must be at a low elevation and internal combustion motors are desired for operaton. It was constructed in 1903 as an alternative to a very expensive sewer for the small number of people to be served until the district was developed considerably beyond its population at that time. The sewage was delivered to a circular tank 18 ft. in diameter and about 7 ft. deep inside, holding about 13,000 gal., Fig. 307. The walls were 12 in. thick, on 18-in. footings, and the bottom was 6 in. thick with a downward slope to a central sump about 1 ft. deep. The tank was divided by a 10-in. wall through the center into two halves, and an 8 × 8-in. sluice gate at the bottom of the wall enabled either side to be shut off for repairs or cleaning. The tank

operate. The force mains consist of 2400 ft. of 8-in., and 2140 ft. of 10-in. cast-iron pipe.

The cost of this installation was approximately as follows: Ejectors, \$2950; machinery, \$1421; air and force mains, \$8555; ejector wells and storage chambers, \$5587; total, \$18,513.

A test of this installation was made on March 20 and 21, 1906, at which time it was found that the efficiency was about 17 per cent., based upon the electric current delivered at the switchboard.

The cost of labor at that time, per million gallons raised 1 ft., was about \$0.68.

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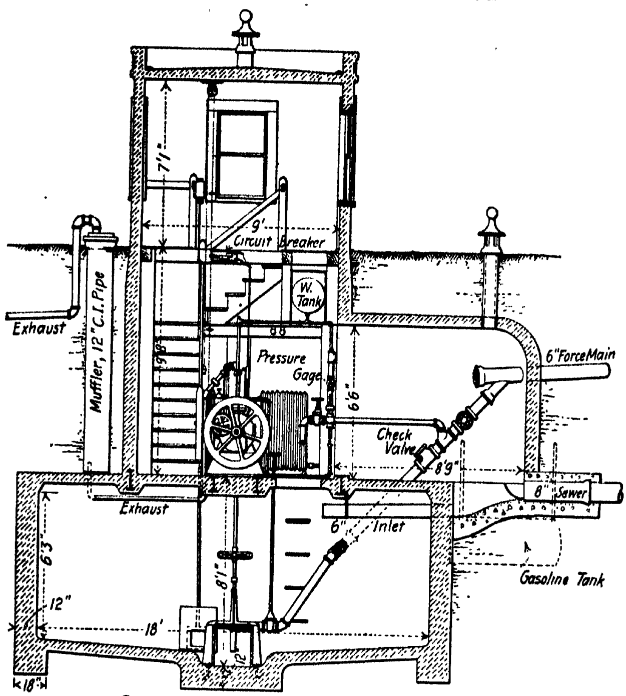
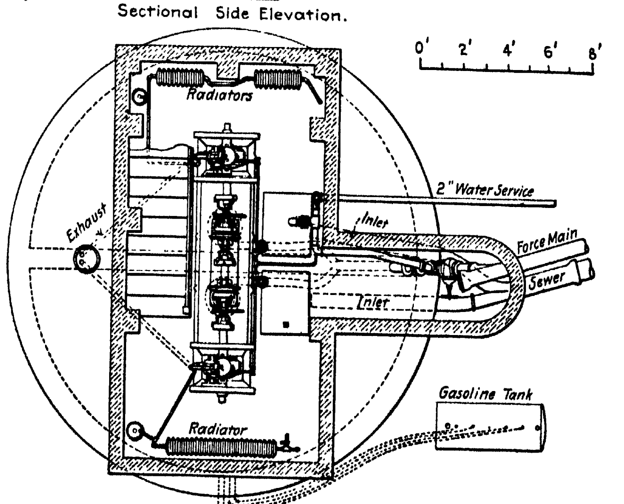
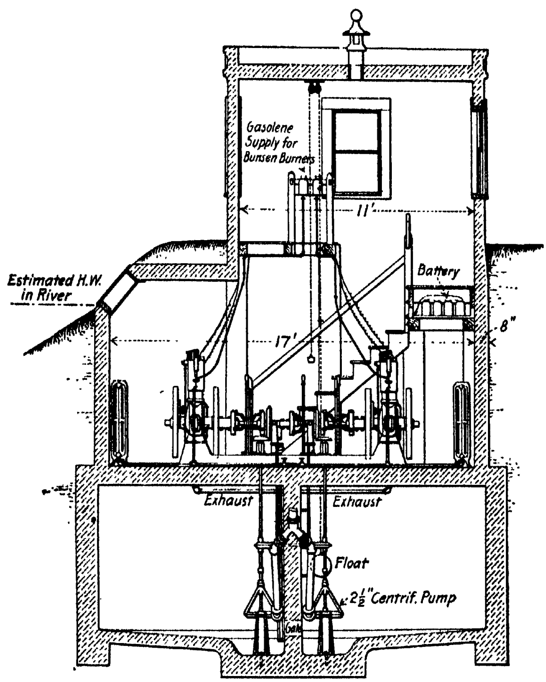


Sectional Side Elevation.



Sectional Plan.

bottom. All pipes through the walls of the pump well enter through iron sleeves with two circular flanges, one on the outer end which is riveted and calked to the shell and the other inbedded in the brickwork to form a cutoff.* The iron pipe passing through each of these sleeves was calked on both ends by means of yarn and lead, like a cast-iron pipe joint. In case the pumps should be out of commission for any reason and the sewage should rise in the wet well, there is an 8-in overflow pipe at about

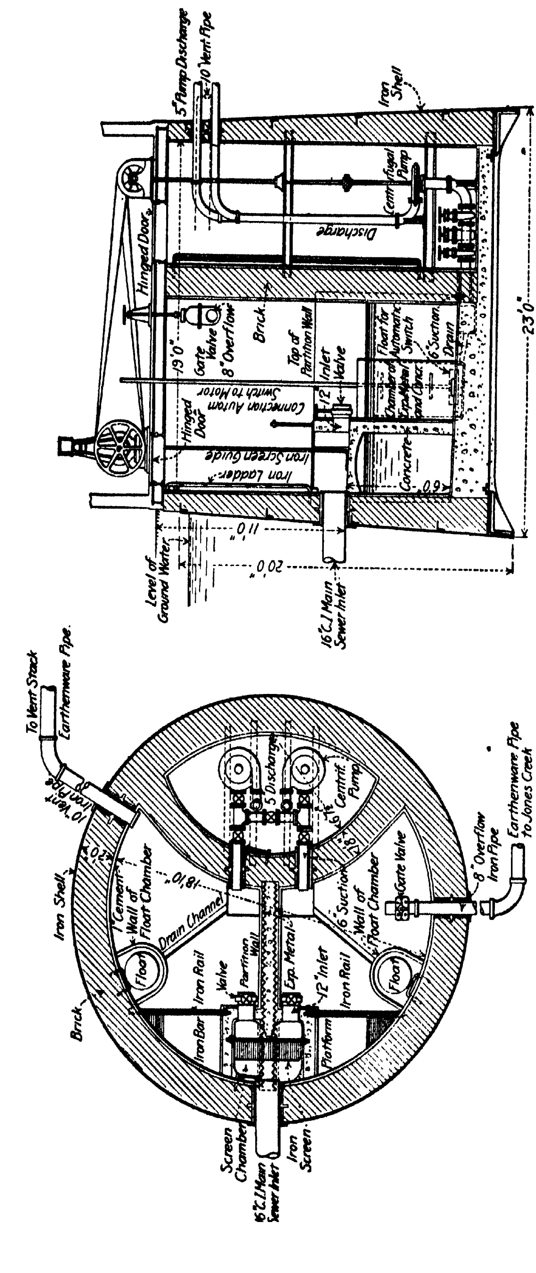


Sectional Front Elevation.

FIG. 308.—Pumping station at Newton.

the ground water level, which will allow the sewage to pass into a creek 90 ft. distant.

Each of the 5-in. centrifugal pumps is driven by a 15-h.p. 3-phase 230-volt 60-cycle induction motor, started and stopped automatically by the action of one of the floats previously mentioned. A rod rising from the float moves a lever connected with a device acting like an elevator controller. Current is obtained from a local electric railway company.



Sectional Plan.

Sectional Elevation.

FIG. 309.—Pumping station at Hampton Institute.

A small engine obtaining steam from the neighboring power house of the institute has been installed as a reserve; it drives one of the pumps through a belt to a pulley on an extension of the armature shaft of the motor. The combined efficiency of the pumps and motor on short runs ranged on test from 52 to 73 per cent., averaging 63.7 per cent.

The pump well is ventilated through a 10-in. pipe running to a ventilating stack, which also vents the tanks to which the sewage is pumped; these tanks and the shaft are described later in this chapter under the head of Storage Basins.

Chicago, Ill.—Sewage pumping at Chicago is carried on in two large stations which are unique in design, owing to the peculiar plan of that city's sewerage system, involving the discharge of crude sewage into the branches of the Chicago River and the reversal of the natural direction of flow in the South Branch, so as to carry the sewage to the inlet of the drainage canal. The 39th Street pumping station, the first which went into operation, was built to pump sewage from a large intercepting sewer along the lake front through a 20-ft. gravity conduit to a fork of the South Branch. In order to dilute this sewage so that it would cause no offense in the open channel of the river after leaving the conduit, arrangements also had to be made to pump along with the sewage a large amount of water from Lake Michigan, so that two sets of pumping machinery became necessary.

The general arrangement of the station is shown in Fig. 310 (*Eng. News*, Sept. 10, 1908). Of the centrifugal sewage pumps two have a capacity of 75 cu. ft. per second against a head of 24 ft., and handle the dry-weather flow; the minimum flow in 1908 was about 90 cu. ft. Each of the two larger pumps has a capacity of 250 cu. ft. per second against a head of 13 ft.; they were installed to handle the storm-water flow, and when this is being done a lift gate at the end of the channel is closed so as to keep lake water from the pumps. Ordinarily this gate is open and the lake water is prevented from reaching the dry-weather pumps by a gate acting like a tide gate.

While the arrangement of the channels leading to the large centrifugal pumps is such that they can be used to pump flushing water from the lake into the outfall conduit, this service is ordinarily performed by two screw pumps, each rated at 666 cu. ft. per second. The maximum head against which these pumps were designed to operate was 7 ft.; it is possible at certain stages of the lake to supply water by gravity from the lake to the conduit, for which purpose a special channel was provided, closed at its entrance by a gate operated like one of the leaves of a lock gate.

The centrifugal pumps are operated by horizontal triple-expansion engines and the screw pumps by vertical triple expansion engines. There are six 264-h.p. water-tube boilers to supply steam.

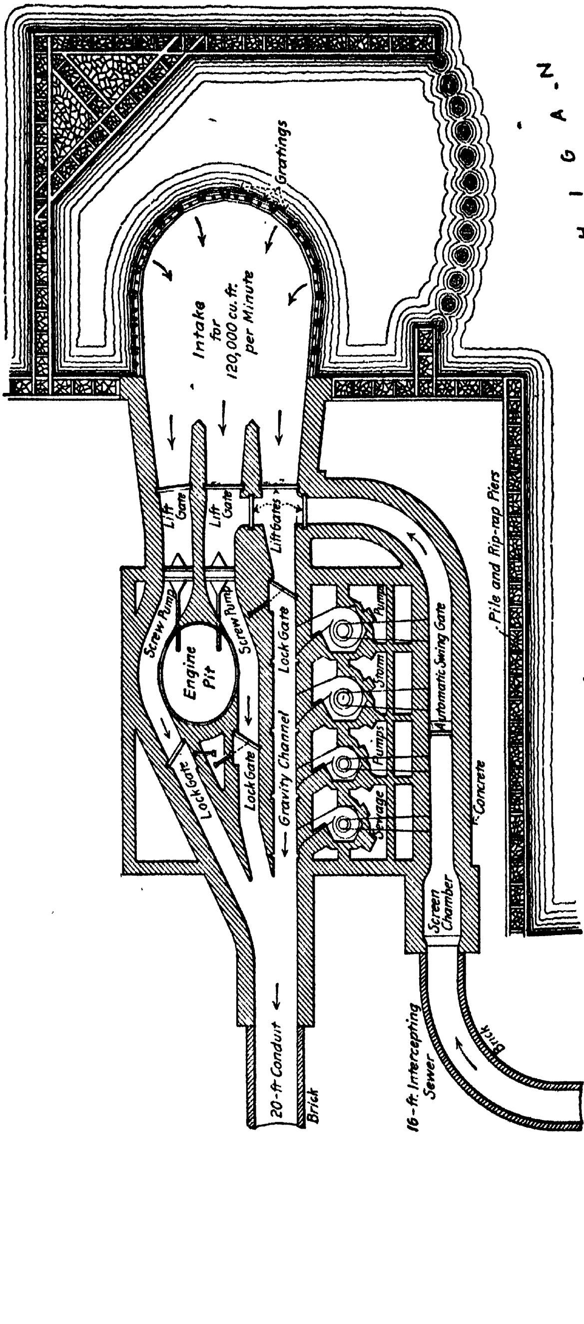


FIG. 310.—Thirty-ninth street pumping station, Chicago.

Dayton, Ohio.—The sewage pumping stations in Dayton, Ohio, attracted considerable attention from designing engineers for some time, on account of the rather unusual control apparatus with which three of them were provided, which has been stated by the local authorities to work very satisfactorily. One of the four stations has two 20-h.p. 3-phase, 60-cycle motors geared to vertical submerged centrifugal pumps with a capacity of 2500 gal. to an average lift of 20 ft. The other stations have two units each. Each unit has a double-suction vertical submerged 4500 gal. centrifugal pump direct-connected to a 40-h.p. 3-phase 60-cycle 2080-volt motor. The starting apparatus referred to is contained

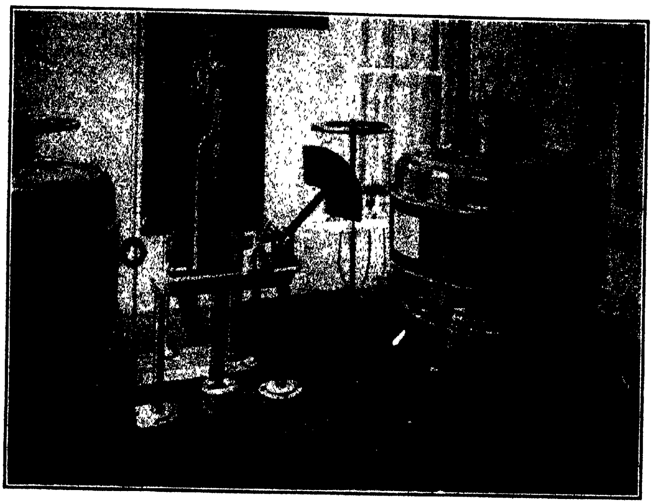


Fig. 311.—Automatic pump controller, Dayton.

in these three stations. As described in *Eng. News*, April 30, 1908, it is worked primarily by float-operated valves in a cylinder which receives water from the city mains. A piston in this cylinder raises the level of an auto-starter to the "starting" position, and at the same time rotates an arm carrying a heavy counterweight, as shown in Fig. 311. About the time the motors come up to speed this counterweight reaches a dead center position and falls over, throwing the lever on the starting panel to the "running" position. As the sewage is disposed of so that the level falls to a predetermined point, the float valves operate the piston to give a reverse motion to the counterweight arm, which in turn brings the starting lever on the panel to the "stop" position, cutting off

current from the motor. The electric equipment of the three stations was furnished by the Westinghouse Electric & Manufacturing Co.

Waltham, Mass.—The sewage pumping station built at Waltham, Mass., in 1907, from the plans of City Engineer Bertram Brewer, has a storage well 19 ft. 1 in. diameter and 17 ft. deep, with plain 10-in. concrete walls, constructed by holding up the sides of the pit with 4-ft. poling boards braced by ribs of 4 to 8 half-inch boards nailed one over the other to complete the circle. The well is about 50 ft. from an adjoining river and below its level except for the upper 2 ft., but it is entirely waterproof, owing to the care taken in selecting and mixing the material and to the use of hydrated lime to increase the impermeability of the concrete. The plant consists of two 5-in. vertical centrifugal pumps direct-connected to 15-h.p. vertical motors, which are started and stopped automatically by means of a Westinghouse controller. The sewage is screened through a basket screen and enters the pumps through very short suction pipes; the pumps are in a dry well as shown in Fig. 312 (*Eng. Record*, March 7, 1908). It was stated in that journal that the plant cost $7000.

The automatic starting and controlling devices for the alternating-current motors of this plant consisted of the usual float and counterweight operating a sheave or hollow drum, a weight on the end of a lever, two spiral springs, and a pawl arrangement for regulating the action of the springs. The operation of the apparatus was described as follows by Mr. Brewer in the article previously referred to.

"A sheave is mounted loosely upon a shaft; an iron ring, cast on the side of the sheave, has a slot cut in it through which passes the arm carrying the weight. This weight-arm is also free to move on the shaft. The slot in the sheave-ring is just long enough to allow the weight-arm to fall from the vertical to the resting place, an arc of 125 deg., so that when the sheave is turned through a distance of 125 deg. the weight will be lifted to the perpendicular and allowed to fall an equal distance in the opposite direction. When the weight falls, the weight-arm engages two spiral springs, which are coiled loosely around the shaft. These in turn rest against a casting which is screwed to the shaft, but which is prevented from turning by a pawl, which is held by notches in the main shaft. The weight-arm compresses the springs, and then trips the pawl and the spring moves to the next notch. The pawl is tripped three times during the downward motion of the weight, and each time it is tripped it allows the shaft to be turned a certain distance by means of the compressed springs, and the shaft in turning operates the auto-starter, throwing it through the three notches to the full position. When the weight falls in the opposite direction, the auto-starter is thrown to the off position. The time of the fall of the weight is controlled by a dash-pot, situated at the end of the weight-drum bearing.

"The float mechanism, while very simple in design, was a source of considerable annoyance at first, owing to the constant variation in temperature

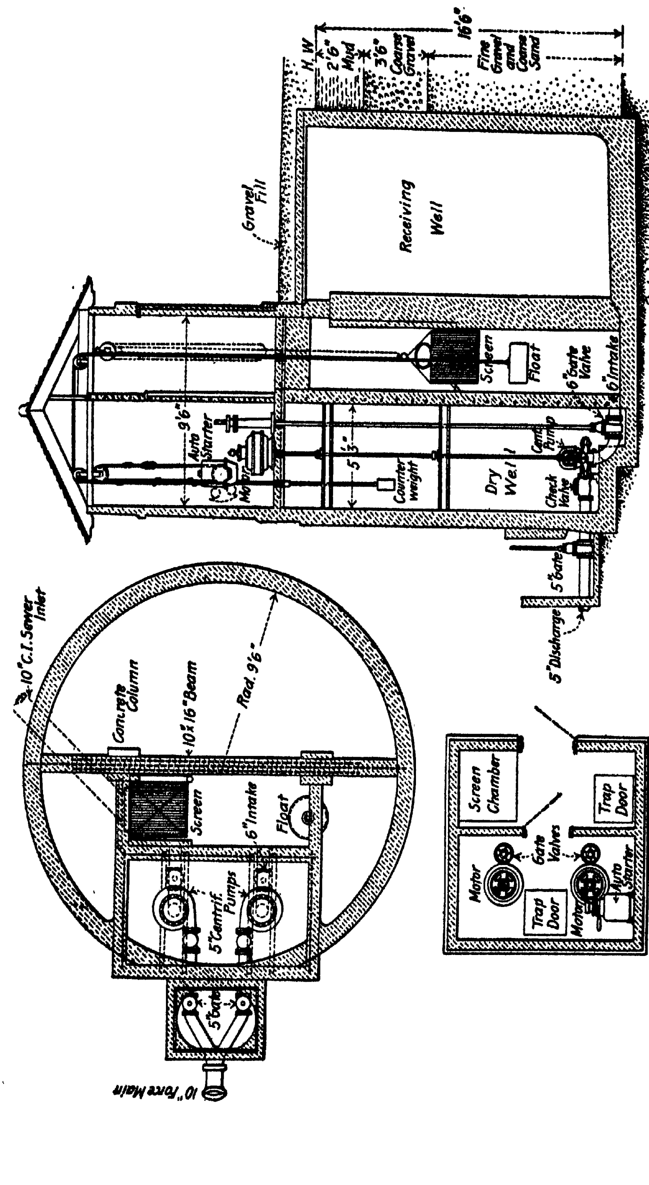


FIG. 312.—Sewage pumping station, Waltham, Mass.

inseparable from an unheated, isolated plant. The control rope was necessarily long and its length was materially affected by changes in temperature, and when too loose or too tight would not operate the controller. The difficulty was overcome by inserting a heavy spring in this control rope. This difficulty surmounted and the weights of float and counter-weight made ample to furnish the necessary power to lift the weight-arm on the controller and overcome the considerable amount of friction in the apparatus itself, the operation has proved reliable under the trying conditions of a severe winter."

Saratoga, N. Y.—In the sewage pumping plants at Saratoga, N. Y., and Hudson, Mass., designed by Frank A. Barbour, time-limit relays were installed to cut out the current automatically in case of stoppage of the motors or burning of the switches. The floats were so set that the first pump started with the sewage at a certain level, and if the inflow was greater than the capacity of this pump the sewage rose to the level where it operated the float governing the second pump. This second pump, coupled to an alternating motor, ran at a constant speed and, starting against a closed check with no discharge, developed the necessary pressure to lift the check and begin pumping. The capacity of each unit at Saratoga was 1500 gal. per minute, with one pump working against a head of 28 ft.; 1200 gal. with two pumps working against a head of 38 ft.; 1000 gal. with all three pumps working against a head of 42 ft.

Hudson, Mass.—At Hudson, the pumps were set in dry wells below the height to which the sewage rose in the adjoining wet well, with suctions laid through the dividing wall into the collecting well. The intention was to have the pumps accessible for repairs and ready primed with each rise of the sewage. Their total capacity was 500 gal. each with two units working against a total head of 35 ft. Considerable trouble was experienced at this place with the stuffing boxes and leakage of air into the pump casings. As a result, the pumps were frequently run submerged in water. This is mentioned to show the importance of insisting upon having a tight pump casing when the pumps are to be placed in a dry well.[1]

Summer St., Boston.—Difficulties like those mentioned in the case of the Waltham plant, are overcome in the Summer St. Station, Boston,

[1] "To prevent air leakage through the stuffing box on the suction head of the pump, there is provided a gland cage within the stuffing box, on each side of which there should be placed about three rings of graphite packing. On the outside of the stuffing box will be found a 1/4-in. pipe tap, which connects to this gland cage. There should be a pipe run from the discharge of the pump and led to the 1/4-in. pipe tap, thus making a water seal in the stuffing box and preventing all air leakage. The gland should be run just as loose as possible, as otherwise the packing is liable to cut the shaft. A small amount of leakage from the stuffing box does no harm, in fact it is an advantage, as it prevents the packing from heating and at the same time keeps the shaft lubricated." Henry R. Worthington. For pumping sewage the ¼-in. pipe should connect with a clear water supply, as it might become quickly clogged if connected with the discharge pipe.

by the use of a controller without flexible cords, as shown in Figs. 313 and 314. The copper float is clamped to the end of a vertical rod and rises and falls in an 8-in. cast-iron pipe, giving it a range of motion of 3 ft. A short distance above the float a heavy rubber ring is attached to the shaft, and the pressure of this ring against its seat, the cap of the float chamber, is sufficient to make a tight joint and prevent sewage from escaping from the chamber.

This station, which was designed by C. H. Dodd, under the general direction of E. S. Dorr, is shown in Fig. 314. The station is underground, with the exception of a narrow concrete entrance hood rising above the sidewalk just inside the curb line. The sewage enters through a 24-in. iron pipe terminating in a sluice gate. There is a bar screen in the gate manhole even though a 10-in. pump might pass anything likely to reach it through the sewers. Provision has also been made for freeing the impeller from rags without dismantling the pump, handholes being provided for the purpose. The sewage is passed through a channel formed by brick side walls running diagonally across the gate chamber, and then enters a wet well 8 1/2 ft. long, 6 ft. high and 2 ft. wide. This is long enough to feed the suctions of three pumps, but for the present only two have been installed. In addition to the three 10-in. suction pipes connections are made from the wet well to the float wells, previously described, and also to a sump from which the sewage and drainage are raised by a water jet ejector. Each pump has its own switchboard, furnished by the Cutler-Hammer Company. The air in the pump room is drawn out through a blower which forces it up to the entrance hood, where it escapes through a grating in the top of the iron door by which the entrance shaft is closed. The motors are carried on a floor supported by 6-in. I-beams; the structure as a whole is built of reinforced concrete carried on piles.

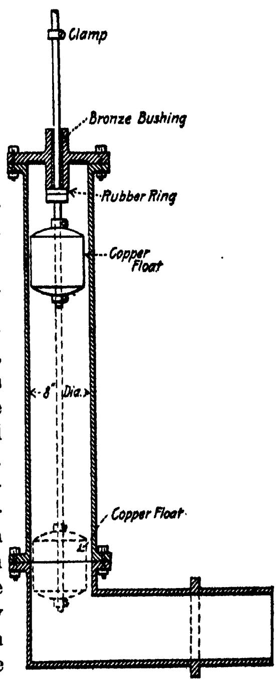


FIG. 313.—Float and float well, Boston.

One of the details of this station is a balanced back-water gate,

Fig. 315. These gates are not designed to act like tide-gates, but are placed on the discharge pipe to prevent water from backing through them and causing trouble when the pumps are taken apart for repair. In order that they may be as sensitive as possible a cast-iron ball is held at the proper position along a rod running up from the gate to counterbalance the latter. It can be adjusted very closely and held in place by a bronze screw, and offers less resistance to the flow of sewage from the discharge pipe than the ordinary type of heavy flap valve. A pair of lugs on each flap and seat permit them to be bolted together when the discharge pipe is to be closed to protect workmen while the pump casing is opened.

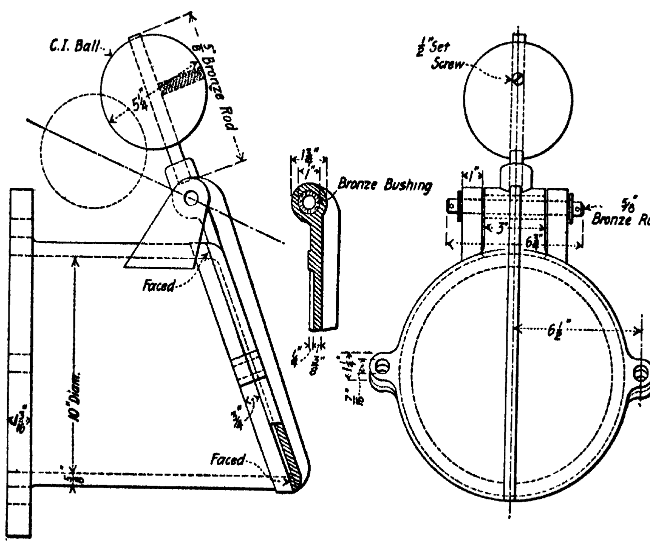


Fig. 315.—Backwater pump-discharge gates, Boston.

Another detail of this station is the cover of a manhole, which had to be large enough to permit machinery to be lowered into and removed from the engine room. For handling the machinery in the room there is a 7-in. I-beam in the roof, from which a hoist is suspended. This runway extends to the manhole which is 3 ft. 9 in. wide and 5 ft. 3 in. long. Inasmuch as it will very rarely be entered, it was considered desirable to offer as little obstruction to travel as possible, and accordingly the Boston standard rectangular frame was chosen. In this case, however, it was also desirable to prevent moisture from accumulating below the iron cover and dropping into the portion of the engine room below it.

This was not unlikely to happen when the weather outside was very cold, for on such occasions the temperature in the pump chamber might be 25° or more warmer than the manhole cover. To overcome this dripping a cast-iron rabbet was placed in the top of the manhole masonry. A wooden cover consisting of 4 × 2-3/4-in. timbers with 2 × 4-in. battens was laid on this rabbet and the edges all around it calked and pitched so as to make a perfectly tight cover, Fig. 316. Between it and the bottom of the cast-iron manhole cover there is a considerable air space, which has prevented any gathering of moisture.

Large Station, Boston.—A much larger station, Fig. 317, built in the same city in 1914, from the plans of the same designer, is probably the largest sewage pumping station with automatic control down to the time of its construction. The building is 65 × 40-1/2 ft. in plan; the

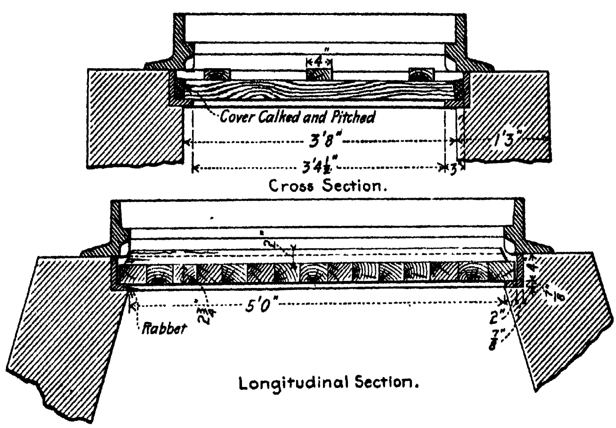


FIG. 316.—Anti-condensation manhole cover, Boston.

basement is much larger. Along one side of the building extends the motor room, and in order that machinery may be moved into and out of it readily there is a large doorway at one end and a return in the curbing, so that a motor truck can be backed into the building for some distance, the floor being strengthened for the purpose. A transformer room in one corner of the building can be entered only through an outside door, the keys to which are in the possession of the employees of the local electric light company, the sewer service having no responsibility for the care and maintenance of the transformers. Adjoining the transformer room is a small room, also entered only through an outside door, affording access to a manhole leading to the

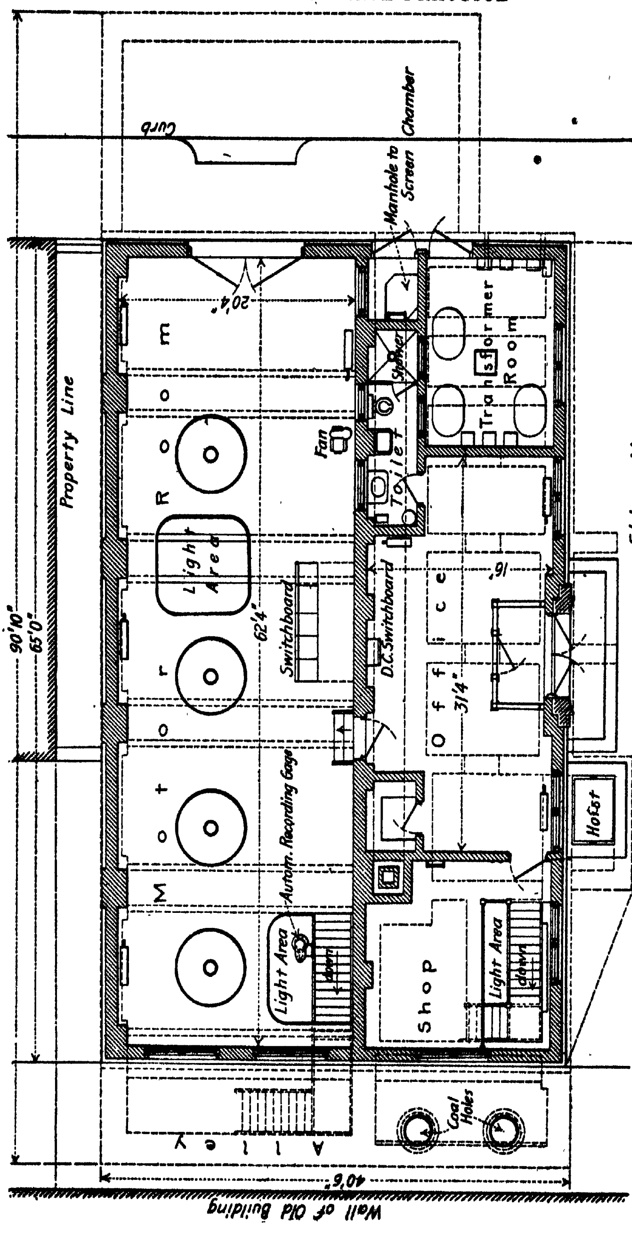


FIG. 317.—Large automatic pumping station, Albany and Union Park Streets, Boston.

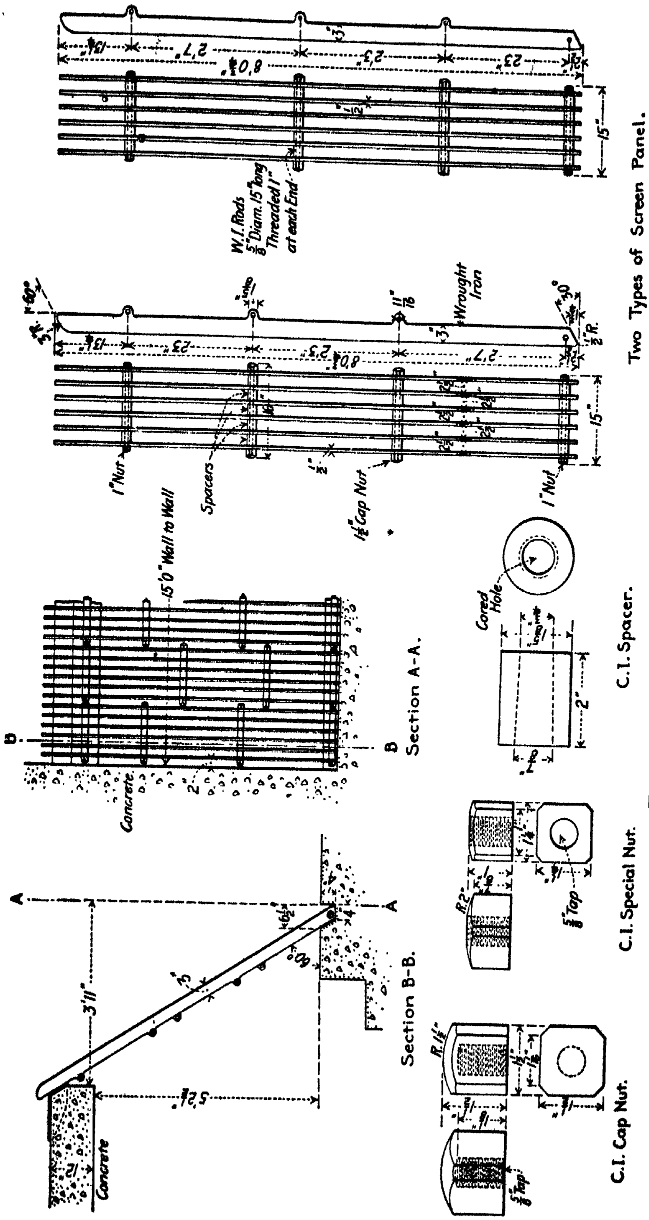
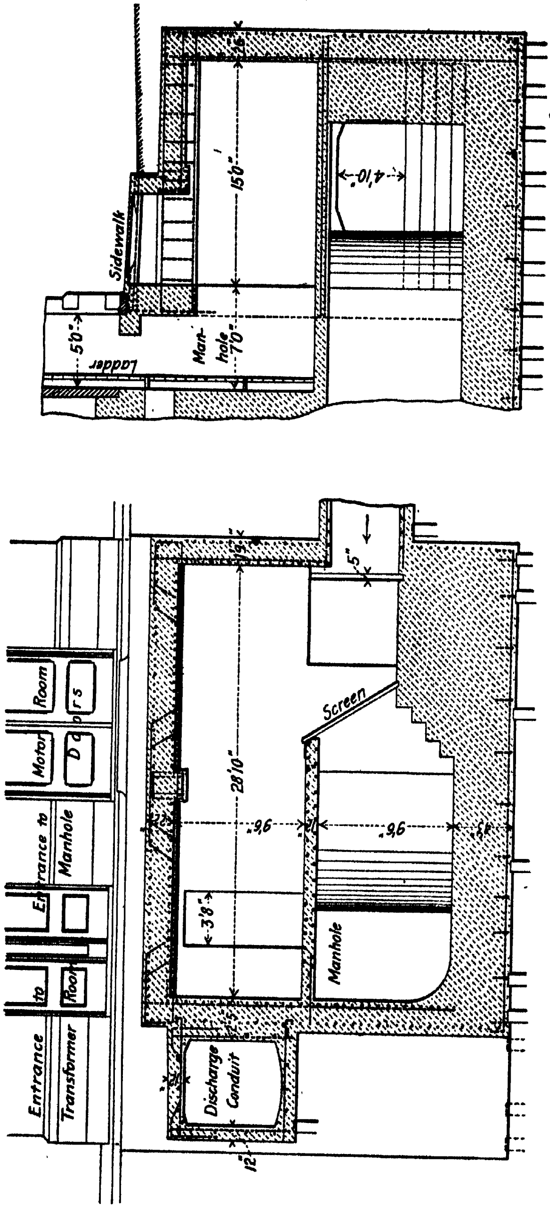


FIG. 319.—Sewage screens used at Boston.



Transverse Section.

Longitudinal Section.

FIG. 320.—Screen chamber, large automatic station, Fig. 317.

screen chamber. The remainder of the ground floor is occupied by an office with a large store closet and by a shop.

The station contains three 150-h.p. motors, each driving a 36-in. centrifugal pump, and a 75-h.p. motor driving a 24-in. pump, Fig. 318. The sewage enters the station through a screen chamber, provided with a screen constructed according to the details shown in Fig. 319. The screen is in twelve panels, each 15 in. wide, and 8 ft. 3/4 in. long. The general arrangement of this screen chamber is shown in Fig. 320.

The pumps are controlled by a float in a well of the type illustrated in Fig. 313, one well sufficing for all pumps, the switch mechanism throwing into service one pump after the other as the level of the sewage in the suction chamber rises. The electric devices for this purpose were furnished by the Cutler-Hammer Co. There is another float well in this station which operates an automatic recording gage, of a type in use in several places on the Boston sewerage system. It was designed by Mr. Dodd and has a pen moved vertically by the float rod over the surface of a chart which is revolved horizontally by clockwork.

The small pump has its suction run into a sump 3 ft. lower than the remainder of the suction chamber, so that this pump can be used to drain the station down to the level of the pump room floor. Below that grade the drainage is removed by hydraulic eductors with suctions in small cast-iron sumps in the concrete floor.

The positions of the 2 1/2-in. bronze nipples and gate valves for blowing off each pump casing and the bottom of each hydraulic gate valve are indicated in Fig. 318. The hydraulic gate valves are connected by 1-in. pipe with the street mains. The end of the discharge pipe has a large backwater gate of the type illustrated in Fig. 298.

Washington.—The sewage pumping station at Washington, D. C., designed by Asa E. Phillips, superintendent of sewers of the District of Columbia, has been much praised by engineers, European as well as American. The general arrangement of it and of the conduits leading to and by it, which form one of its most interesting features, is shown in Fig. 321, from *Eng. Record,* Aug. 29, 1908. At this station the entire sewage of the city is pumped through a pair of 60-in. pipe about 18,000 ft. long to a point in the Potomac River about 800 ft. from shore. The large conduits on either side of the station discharge into the Anacostia River, on the bank of which the station stands, the storm water from a considerable part of the low-lying portion of the city. A part of this storm water is discharged by gravity, while another part must be pumped at certain stages of the river.

The Tiber Creek and Jersey Avenue high-level intercepting sewer passes along the east side of the pumping station. Before it reaches the station its lower portion has a section 14 ft. wide and 14 ft. 3

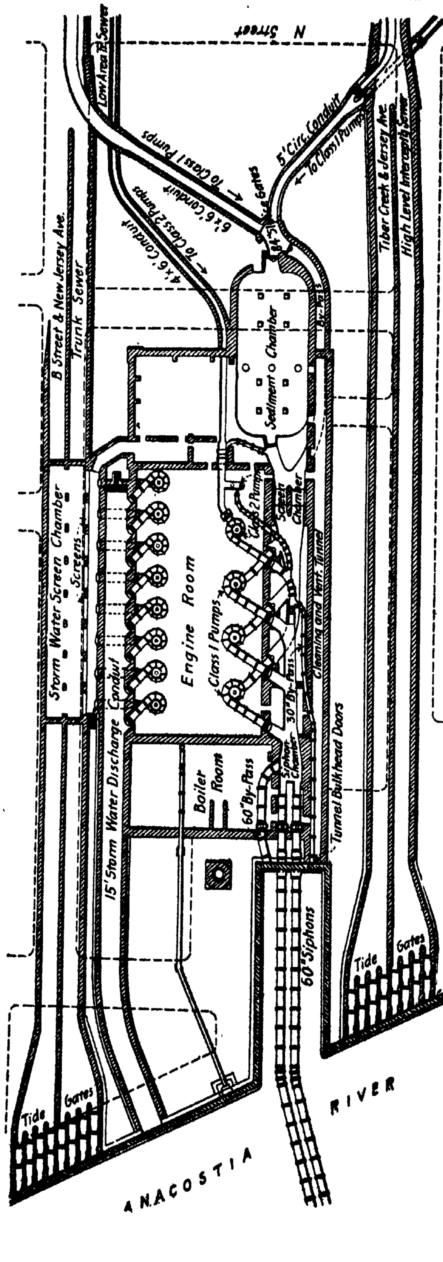


FIG. 321.—Plan of pumping station at Washington.

in. high, with a cunette, or dry-weather channel, diverted near the station, into a 5-ft. circular conduit, into which the east side intercepter 6–1/4 ft. in diameter, also discharges. Beyond the point where the dry-weather channel is led to one side, the Tiber Creek sewer continues as a twin section, each channel being 12 ft. wide by 10–1/2 ft. high, the invert level with the berm of the cunette section. On the west side of the pumping station the B Street and Jersey Avenue trunk sewer extends. This also has an 18-ft. cunette section 16 ft. high, before it reaches the pumping plant. Where the dry weather channel, or cunette, is diverted to one side, the main sewer becomes a twin section, each side being 12 ft. wide and 10–1/2 ft. high. All these sewers are built of concrete with a lining of vitrified brick on the portion of the invert subject to greatest wear and red brick on the other parts of the invert over which sewage is likely to pass at some time.

The diversion conduit for the dry-weather sewage from the B Street sewer, is 6 × 6 ft. in size and joins the 5-ft. circular conduit from the Tiber Creek sewer at a gate chamber containing two 84-in. sluice gates. One of these admits the sewage, during the normal operating conditions, into a sediment chamber 50 × 104 ft. in plan, having a groined arch roof carried by columns 3 ft. square and 16 ft. apart in the clear. This chamber extends partly under the pumping station and is large enough to reduce the rate of flow of the sewage to considerably less than 1 ft. per second. The sediment which is collected in the chamber is removed in 2/3-cu. yd. buckets. These are brought into the chamber on cars run into it on an industrial track laid on the floor, and are filled by hand. The cars are run under a hatch in the roof and the buckets are lifted from them to a trolley on an overhead track at a much higher elevation, by which they are transferred to the river, where their contents are dumped into a barge. The overhead track runs for part of its length through an 8 × 8 ft. passage or tunnel, which is also used as a part of a system of ventilation worked out so completely that no offensive odors have been detected about or in the station.

The sewage is drawn from this chamber into an 8-ft. conduit having a check gate and a twin screening chamber. This screen chamber is 30–1/2 ft. long, 20–1/2 ft. wide, and divided into two equal portions, each with two screens of 3/4-in. rods on 2–1/4-in. centers, operated by hydraulic cylinders and counterweights. The trash from the screens is removed through a branch connection with the conveying and ventilating tunnel just mentioned. The sewage passes through this screening chamber into a suction chamber, from which three centrifugal pumps draw their supply. These lift the sewage into a 16 × 22 × 40-ft. siphon chamber at the head of an inverted siphon under the Anacostia River, which forms the first part of the outfall sewer. The gate valves on the head

of the two pipes forming the siphon have their seats on the downstream side cut away so as to leave no bottom slot in the valve bodies in which sediment can be collected. In case the sediment chamber is out of service for cleaning, a by-pass delivers the sewage from the gate chamber directly to the pumps. The latter are known as Class I pumps, to distinguish them from two of smaller capacity installed for a special purpose. The sewage from a small low-lying district served by a separate system, independent of the trunk and intercepting sewers, is delivered through a 3-1/2-ft. sewer which has no connection with the settling basin, but runs directly to these smaller pumps known as Class II, an arrangement necessary to obtain proper hydraulic gradients. The pumps discharge the sewage into the siphon chamber or through an emergency by-pass into the river. There is a screen chamber in the suction conduit of these pumps, and a by-pass is provided so that either Class I or Class II pumps can temporarily be used for the service of the other.

The storm water delivered through the Tiber Creek sewer passes directly into Anacostia Creek through the tide gates on the bulkhead, as indicated in Fig. 321. The storm water brought down by the B Street and New Jersey Avenue sewer must pass first, however, into a storm-water chamber, 160 ft. long, 36-1/2 ft. wide and 16 ft. high, having a roof of concrete arches carried by I-beams. Along one side of this chamber are openings fitted with screens of 1-1/2 in. wrought-iron pipe on 4-1/2-in. centers, placed on an inclination of 1 to 6. An elevated platform between the walls of this chamber and the pumping station has been constructed for use in cleaning the screens. When the elevation of the water in the river permits, the storm water passes directly through this chamber into the river. When the latter is high, however, tide gates prevent a backflow into the conduit and the storm water that comes down is pumped from the chamber into a 15-ft. discharge conduit at a considerably higher level, eight pumps being provided for this especial purpose. It will be observed that it is also possible to utilize the Class I pumps for handling some of this storm water, in case of emergency.

The pumping station has at the land end a three-story 75 × 120-ft. section used for office and shop purposes; in the middle there is a 90 × 170-ft. engine room, and on the river front a 60 × 120-ft. boiler house with elevated coal bunkers. The Class I pumps are three in number, each driven by triple-expansion engines and rated at 100 cu. ft. per second to a height of 27 ft. One of them is a reserve. There are two Class II pumps, one a triple of a capacity of 32 cu. ft. per second raised to a height of 29 ft., and the other a compound of equal capacity. The storm-water pumps discharge under a variable lift; each is capable of raising 100 cu. ft. per second to a maximum height of 15 ft., but they

are particularly effective at their usual lift of 3 to 8 ft. Owing to the fact that they are in operation only a portion of the time, they are driven by compound engines. All engines but one are of the horizontal type, without fly-wheel, direct-connected to a vertical pump shaft, first developed by the Allis-Chalmers Co. for one of the Boston sewage pumping stations. The Washington pump setting differs from that of earlier stations in the omission of separate chambers for each pump, for in Washington the entire basement of the engine room serves as a large dry well. The only vertical engine is the compound driving one of the Class II pumps, a unit which had been used during the construction of the station, and was in good enough condition to be installed as a reserve in the permanent plant.

The engines are supplied with steam by six water-tube boilers, each of 275 h.p., with automatic stokers, fuel economizer, complete mechanical coal handling machinery, and the other accessories and auxiliaries of a high-grade power plant.

Baltimore.—The Baltimore sewage pumping station is provided with a main engine room 180 ft. long, 54 ft. wide and 68 ft. high from the basement floor to the chord of the trusses. Eventually it will contain five pumping engines, two drainage pumps, a 20-ton electric crane, an electric switchboard, and valves and piping. Three pumping engines have been installed, which were built by the power department of the Bethlehem Steel Co. These are of the vertical, triple-expansion, crank and fly-wheel type, Fig. 322, rated at 27,500,000 gal. in 24 hours against a head of 72 ft. when the speed is 20 r.p.m. The pump has three single-acting plungers, 40-1/4 in. in diameter and 60 in. stroke, and the valve chambers have very large clack valves shown in Fig. 301. Each engine is rated at about 400 h.p. at normal speed. On test the average duty was 165,000,000 ft.-lb. per 1000 lb. of dry steam, with an average slip of about 3-3/4 per cent. The drainage pumps are 12-in. centrifugals driven by 40 h.p compound condensing engines and have a capacity of 3000 gal. per minute each. They draw their supply from the underdrains of the low-level sewers, and discharge it through the condensers of the sewage pumping engines or directly into the harbor. Between the engine room and the boiler room is a screen chamber where the sewage is first sent through movable coarse screens and then through finer fixed screens over the ends of the suction pipes. The boiler room is 94 × 50 ft. and contains space for five water-tube boilers, each of 265 h.p. At the present time three have been installed, together with one of the two economizers for which space is furnished, and the coal and ash handling machinery and various auxiliary machinery.

Providence.—The sewage pumping station at Providence, R. I., is particularly interesting because of the reconstruction of the plant in

1911. The original plant contained Holly engines installed in 1896, of the direct-acting triple expansion condensing type, each unit having

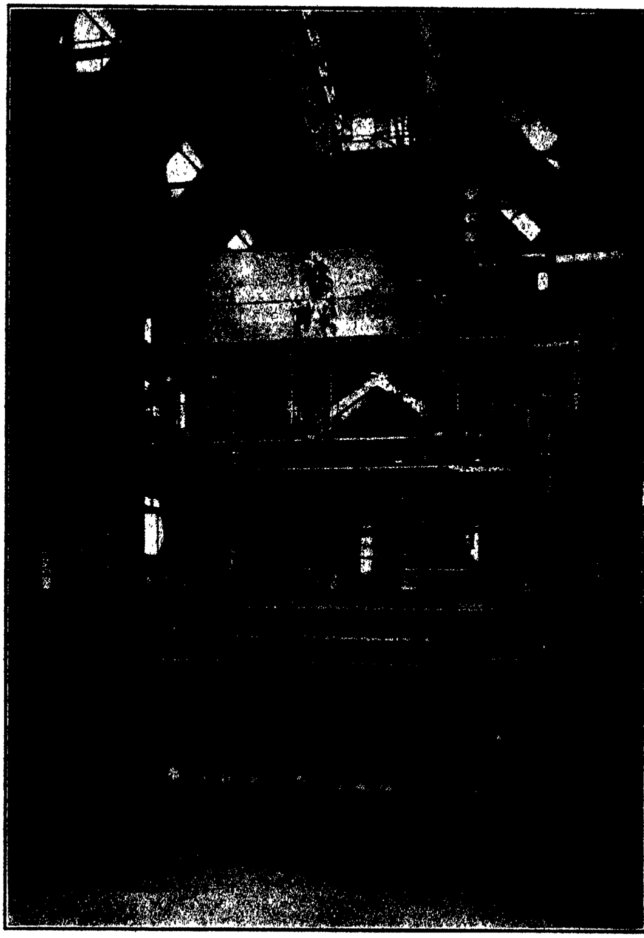


FIG. 322.—Interior of Baltimore pumping station.

two fly-wheels. The pump plungers were 42 in. in diameter and 8 ft. long. The sewage valves were of the weighted clack type, having a rubber disk 5/8 in. thick opening 1-7/8 in. The deterioration of the

valves caused much trouble. The valve seats were bushed with bronze and became badly worn, so that the valve disks had a short life and oftentimes became broken off. While repairs were being made heavy storms sometimes occurred in summer, resulting in the flooding of the lower parts of the station with storm water and sewage. These conditions were considered so unsatisfactory that it was finally decided to abandon the old water end entirely and to substitute for the plunger pumps under each engine, two centrifugal pumps driven by ropes from the two flywheels. Ropes were chosen rather than belts, because of the possibility that the pump wells might be flooded, which would cause excessive slipping of the belts. The reconstruction was accomplished by bolting circular rings to the flywheels, each containing 11 grooves. The old water end was removed entirely, leaving ample room for the two centrifugal pumps, each of the capacity of 15,000 gal. per minute against a total operating head of about 31 ft. The capacity of each engine has been increased about twenty per cent. by this change, the total cost of the reconstruction of the three units being about $38,000.

Batavia, N. Y.—There is a small steam driven station at Batavia, N. Y., where a horizontal 100-h.p. Corliss engine drives a vertical centrifugal pump rated at 470 gal. per minute and two rated at 1050 gal. each against a total head of 64 ft. City Engineer Robert L. Fox informed the authors that this system of driving was accomplished by rope drives between the engine and the main shaft and between the shaft and each of the pumps. The quarter twist necessary to drive a vertical shaft from a horizontal one was readily accomplished with the Dodge rope transmission system.

Detroit.—The pumping station at Detroit, built in 1912, is one of the latest steam-driven plants and has proved so satisfactory in operation, according to information furnished by City Engineer R. H. McCormick, that no changes in any details would be adopted in a new station for the same service. The external appearance of the plant has been favorably criticised and is shown in Fig. 323. The building is 200 ft. long and 50 ft. wide, constructed of buff brick with terra-cotta trimmings above the grey sandstone base courses. The steel roof supports reinforced concrete slabs on which red tiles are laid, and there is a long monitor for ventilation. Inside, the wainscot is of white glazed terra-cotta with gray faced brick above it. The floor is paved with red tile. The doors and window sash are steel.

A 9-ft. sewer enters one corner of the building and discharges into a wet well 130 ft. long and 9 ft. wide, running alongside the pump well. The suction pipes leading from this well have hydraulically operated sluice gates at the ends, and a 24-in. vitrified pipe line leads from the wet well to the base of the stack for ventilation. For handling the dry-

weather flow there is a 24-in. centrifugal pump of the vertical type, with a 150-h.p. motor to operate it. The sewer has such a large capacity that it affords enough storage during dry weather to enable the motor to be shut down during the period of peak load carried by the electric company furnishing current. The storm water is handled by a pair of 42-in. vertical centrifugal pumps, each rated at 100 cu. ft. per second. The small pump is rated at 30 cu. ft. against the same head. Room is left in the station for the installation of another large centrifugal. Each of these storm-water pumps is driven by a horizontal compound condensing engine rated at 542 h.p. with the cylinders placed at right angles with each other. The pumps are located in a dry well which is ventilated by means of an exhaust fan. The thrust bearing is about half way up the shaft from the pump to the crank. Steam is furnished by two 300-h.p. water-tube boilers with automatic stokers. The boiler

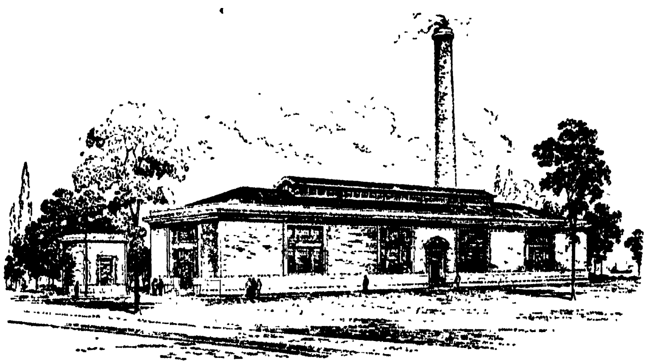


FIG. 323.—Screen chamber and pumping station, Detroit.

settings are finished with white enameled brick. About 200 tons of coal can be stored in bunkers formed by a wall 12 ft. high and 8 in. thick, running along one side of the boiler room. The stack is 120 ft. high and 5-1/2 ft. in diameter.

Lebanon, Pa.—The sewage pumping plant at Lebanon, Pa., designed by James H. Fuertes, of New York, has an unusual system of control, which is now (1914) being duplicated at Dallas, Tex., in a plant designed by the same engineer, the original installation having proved entirely satisfactory in service. The pumping plant was built to deliver sewage to trickling filters under a sufficient head to secure satisfactory results. Two pumps are used, each a volute centrifugal with 8-in. suction and 6-in. discharge pipes, a closed impeller, and guaranteed to deliver 1,000-000 gal. in 24 hours through a total lift of 6 ft., and to overcome a total

lift of 8 ft. when starting in operation. They have horizontal shafts and are directly connected to 6-pole induction motors wound for 3-phase, 60-cycle, 220-volt current. Each motor was required to be able to stand an overload of 25 per cent. for 2 hours without injury. Such pumping units run at constant speed, and consequently some method of controlling their operation was necessary, in order to carry out the desire of the designer to send the sewage to the trickling filters from an Imhoff tank at the same rate at which it reached the sewage treatment works.

It will be seen from the diagram, Fig. 324, that screened sewage is delivered into the Imhoff tank, from which it is drawn by the pumps and discharged through an overhead connection which descends just before leaving the pumping station building on its way to the filters,

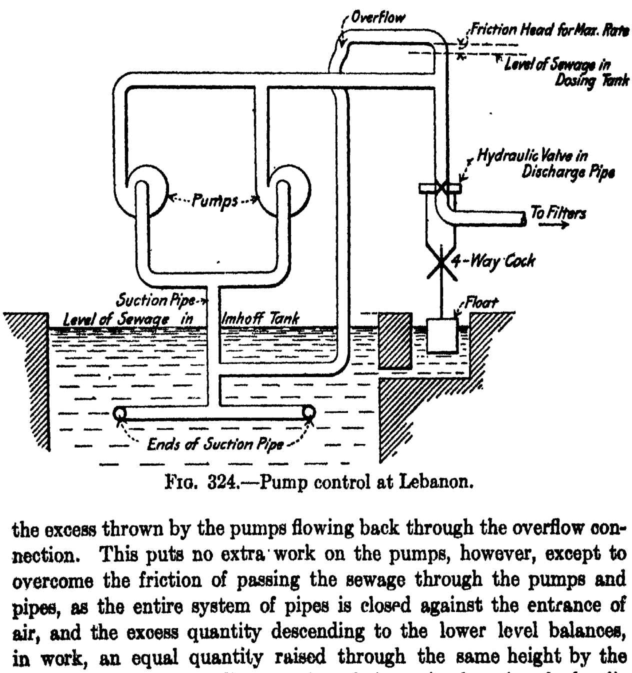


FIG. 324.—Pump control at Lebanon.

the excess thrown by the pumps flowing back through the overflow connection. This puts no extra work on the pumps, however, except to overcome the friction of passing the sewage through the pumps and pipes, as the entire system of pipes is closed against the entrance of air, and the excess quantity descending to the lower level balances, in work, an equal quantity raised through the same height by the pumps. In this descending portion of the main there is a hydraulic valve and at the top of the vertical main there is an overflow pipe which runs back to a connection with the suction main of the pump. The hydraulic valve is opened and closed by pressure water admitted to one end or the other of the actuating cylinder, by means of a four-way

cock. The water supply is taken from the city mains. The four-way cock is operated by a float in a chamber which is connected with the Imhoff tank by means of a 1-in. pipe, so that the level in this chamber is always the same as that in the tank. If the rate of sewage flow from the

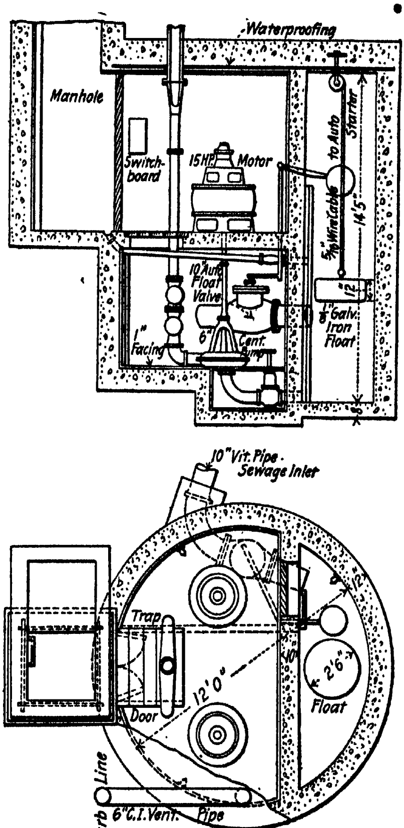


FIG. 325.—Pump chamber with automatic inlet.

city increases, the level of the sewage in the Imhoff tank will rise slightly, and the hydraulic valve will then be opened to a corresponding extent, permitting a larger discharge of sewage to go to the sprinkling filters. If the rate of flow from the city is smaller, the tendency will be for the sewage to fall in the Imhoff tank, thus allowing the float to drop some-

what and turn the four-way cock so as to close the hydraulic valve proportionately thus sending a smaller quantity of sewage to the sprinkling filters.

In this system of control the overflow pipe must connect with the suction pipe below the level of the sewage in the Imhoff tank in order to have all the pipe-ends trapped.

Ridgewood, N. Y.—In most sewage pumping stations of small size reliance is placed upon the automatic starting and stopping devices to prevent sewage rising above a predetermined level, and in case of any accident to the machinery, an overflow pipe allows the excess sewage to escape. In a temporary plant at Ridgewood, Borough of Queens, N. Y., an overflow pipe could not be provided and consequently an automatic shut-off valve was installed. The plant is shown in Fig. 325, from *Eng. Record*, July 24, 1909. There is no screen in the plant, because the sewage is screened through No. 12 galvanized iron 1/4-in. mesh screens at the head of the pipe supplying the pumping station. This inlet pipe has a 10-in. automatic float valve operated through a system of levers by a large ball float in the wet well. Ordinarily this float is not reached by the surface of the sewage, for the automatic starting apparatus throws the pump into service before the sewage reaches the elevation of the ball float. There are two pumping units, each consisting of a 6-in. centrifugal pump driven by a 15-h.p. induction motor working on a 60-cycle, 220-volt circuit, the starting and stopping being controlled by Westinghouse apparatus.

Salt Lake City.—The use of gearing between a small pump and its engine or motor is by no means obligatory and some pump manufacturers have expressed a preference for belts under certain conditions, although the tendency of the belts to slip keeps down the speed of the pumps and belted pumps have a somwhat lower mechanical efficiency than direct-driven pumps on that account and also because of the side pull on the bearings. A plant of this type is shown in Fig. 326. It was built in 1907 at Salt Lake City, from the plans of Lewis C. Kelsey, and operates against a static head of 34 ft. and a total head of 65 ft. The 40-in. sewer terminates in a drop manhole which has a valved opening into each part of a 30 ft. square pump pit, divided by transverse and longitudinal cross walls into four fairly equal compartments, two of them serving as wet wells and two as dry wells. This arrangement permits the use of horizontal pumps, which are generally considered somewhat easier to operate. The driving shaft to which the pumps are belted has a 150-h.p., 60-cycle, 3-phase, 440-volt induction motor at one end and a 150-h.p. 3-cylinder suction gas engine at the other, each connected to the shaft through a friction coupling. The gas producer is rated at 200 h.p. and was designed to use anthracite, but some fairly successful experiments were made with a mixture of coke

and anthracite. The plant has the usual purifier, scrubber, tar extractor, and blower auxiliaries. The producer gas engine is generally run during the period of peak load on the electric company's lines, when an extra price is charged for current.

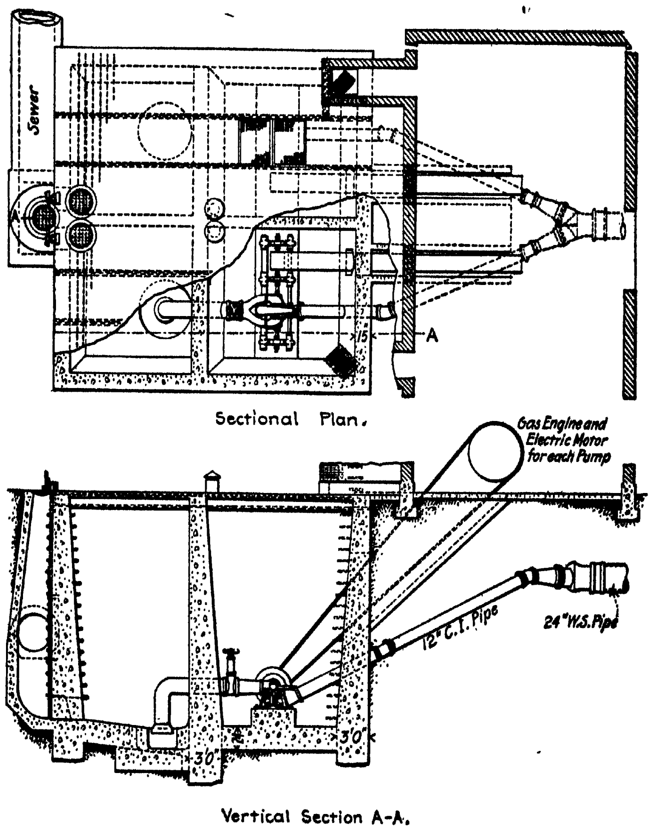


Fig. 326.—Belted pump drive, Salt Lake city.

Kansas City, Mo.—In Kansas City, Mo., there is an area of 500 acres along the Kansas River, from which it is protected by a levee, occupied by important business and manufacturing companies. Owing to the industrial wastes from them, the dry-weather flow from this district is 30 to 50 cu. ft. per second. The trunk sewer is 10 ft. in diameter, and its outlet is so located that the sewage can be discharged by gravity

during most of the year. For about a total of one month, however, the river stage is such that pumping is necessary, for which purpose a gate-house has been constructed on the sewer, and a pumping station has been built upon a by-pass around the gate, the portion of the sewer below the gate forming the outlet conduit from the pump. Owing to the intermittent service, efficiency was not considered important in the design of the plant. Each of the two contrifugal pumps is of the constant speed vertical motor-driven type, with a rating of 30,000 gal. per minute against a head of 22 ft. In a description of the station in *Eng. Record*, Feb. 22, 1913, O. L. Eltinge, who was connected with the design and construction of this plant under City Engineer L. R. Ash, states that pumps of the vertical shaft type were selected, so that the motors could be direct-connected and still be above the flood level, thus preventing damage to them in case of flooded pump pits. As the plant

TABLE 172.—INFORMATION REGARDING STEAM PUMPING STATIONS

City	Baltimore	Deer Island, Boston	Columbus	Detroit	Elmhurst, (Queens Boro')	Far Rockaway (Queens Boro')	Woonsocket
Building							
Length, ft.	188	230	56	200	135	125	53
Width, ft.	157	35	41½	60	125	110	30
Height to eaves, ft.	59	24 & 15	22	22	30	15	16
Height to ridge, ft.	85	....	38½	38	......	25	27
Material	Brick	Brick	Brick	Brick	Brick	Brick	Brick
Boilers, No.	3	6	3	2	2	3	2
Type	Water tube	Scotch & tubu.	Water tube	Water tube	Tubular	Tubular	Tubular
Total h.p.	795	....	450	600	150	180	100
Engines, No.	3	4	5	2	2[1]	2	2
Type	Vertical triple expan.	Corliss triple horiz.	Vertical h.-s. comp.	Corliss horiz. comp.	Worthington	Snow duplex	High speed
Total h.p.	1200	850	400	1084	......	....	70
Pumps, No.	....	4	3; 2	1[2]; 2	1[1]	2[3]	2
Type	Plunger	60"; 42" Cent.	20"; 12" Cent.	24"; 42" Cent.	6" Cent.	6" Cent.	8" Cent.
Total m.g.d.	82.5	235	14	26	2.25	4	3
Suction lift, ft.	5 to 8	0	13.5	32.3	9	14	20½
Static head, ft.	58	19	5	....	8	6	......
Total head, inc. friction, ft.	72	19	20.9	....	......	....	......
Cost of land	$121,022	....	....	$20,000	......	....	......
Buildings	401,175	$85,000[4]	$54,720	135,520	$90,000	$125,000	
Machinery	551,473	$89,000	$131,840	121,500		30,000	$10,400
Engineering	....	....	8.98%	12,401	9.3%	....	

[1] One of the Worthington pumps was removed about 1913 and the motor-driven centrifugal mentioned in the table was put in its place. [2] The small pump is driven by a 150 h p. motor and handles the dry-weather sewage. [3] The two motor-driven centrifugals were installed to handle the extra amount of sewage, when there is a large temporary population at this seaside resort. [4] Without foundations.

TABLE 173.—INFORMATION REGARDING ELECTRICALLY OPERATED PUMPING STATIONS

City	[1] Dayton, Ohio	Grand Rapids, Mich. No. 1	Grand Rapids, Mich. No. 2	Grand Rapids, Mich. No. 3	Grand Rapids, Mich. No. 4	[2] Hartford, Conn.	Kansas City, Mo.	Lake Charles, La.	Minot, N. Dakota	[3] Salt Lake City	Sturgis, Mich.	Toronto, Ont.	Waltham, Mass.	Warren, Pa.
Building	Brick	Brick	Concrete	Concrete	Brick	Brick	Brick	Brick	Concrete	Brick	Brick	Brick	Concrete	Brick
Length, ft.	26	33	57	62	140	70	38	16	13	48	12	111	11	25
Width, ft.	18½	27	31	36	39	42	15	16	13	22	12	65	11	14
Height to eaves, ft.	8	31¼	21	20	31	19	16	19¼	10	...	8	30	10	9
Height to ridge, ft.	16	31¼	32	32	31	19	20	19¼	10	...	11	30	13	16
Motors, No.	2	2	3	3	4	3; 1; 1	2	2	1	2	2	3	2	2
Type	A.C.	A.C.	A.C.	A.C.	A.C.	A.C.	A.C.	A.C.	A.C.	A.C.	A.C.	A.C.	A.C.	D.C.
Total h.p.	80	60	180	150	700	405	250	...	...	300	10	350	30	30
Pumps, No.	2	2	3	3	4	3; 1; 1	2	2	1	3	2	3	2	2
Type	Vert. cent.	Hor. cent.	Hor. cent.	Hor. cent.	Hor. cent.	Vert. cent.	Vert. cent.	Vert. cent.	Vert. cent.	Hor. cent.	Vert. cent.	Vert. cent.	Vert. cent.	Vert. cent.
Total cap., m.g.d.	12	26	63	60	218	70; 4.3; 0.3	43	1.26	1.4	20.5	0.7	37.8	...	1.5
Suction lift, ft.	...	12.7	20	21	22	...	0	7	0	0	0	0	0	0
Static head, ft.	20	7.6	6.7	6.7	10.4	0–20[4]	0–20	13	...	35	...	18	29	50
Total head, inc. fric. ft.	...	...	...	...	...	...	0–30	...	...	68	25	23	...	60
Cost of land	...	...	...	...	...	$10,000	...	...	$800	$1,750	...	...	...	...
Buildings	$3,140	$7,255	$16,581	$11,504	$34,212	33,300	$43,000 (buildings and machinery)	$1,500	5,500	15,000	900	$72,000	$3,871[6]	$1,465
Machinery	4,500	8,315	13,441	16,454	33,396	28,800		1,200	980	17,500	700	33,600	3,000[7]	2,020
Engineering	300	...	...	...	...	...	...	5%	250	1,500	80	...[5]	300	175

[1] There are four pumping stations in Dayton, all practically alike. [2] The Hartford station has three storm-water centrifugals, one for sewage and one for sump drainage. [3] The Salt Lake City Plant contains also a Fairbanks-Morse suction gas producer plant and 150-h.p. engine, which takes the peak of the load as described in the notes on this station; the cost figures include the gas plant. [4] All except smallest pump rated at 16,5′ head; 4″ pump at 20′ head. [5] Engineering not kept separate. [6] Includes piping and 630′ of 10″ force main. [7] Includes recording apparatus and 2 miles of wiring.

stands idle most of the time, it was also considered advisable to have the motors as far from damp places as possible. The pumps were placed low enough to allow them to be self-primed when the valves on their suctions are opened. Each pump is in a separate pit, to enable repairs to be made on either of them without interfering with the operation of the other. As the dry-weather sewage is so small that one pump can handle it in a brief time, it was necessary to provide check valves on the outlets to prevent a reversal of flow when pumping was stopped.

The method of pumping in such cases is as follows: After the gate is closed, the sewage is allowed to pond up behind the gate until the water surface nearly reaches the crown of the sewer, which takes about half an hour. Then one pump is run for about ten minutes, when the ponding of the sewage is repeated again. When the pump is started the first draft nearly empties the sewer near the intake, and it takes nearly a minute for the nearly stationary water at a distance from the station to get into full motion toward the pump. Another feature of the plant, which has been shown by experience to be unsatisfactory is the absence of gratings over the pump intakes, for heavy timbers have entered them; it was proposed in 1914 to install such gratings to remedy this condition.

Some information regarding pumping stations of a great variety of types is given in Tables 172 and 173. The statements were furnished by interested city officials in every case but one, and are doubtless as comparable as such information ever is. The statistics were furnished by about a third of the engineers to whom inquiries were sent, and the authors are particularly grateful to them for supplying records that have heretofore been unavailable for the use of most people.

ECONOMIC SIZES OF FORCE MAINS

A problem of frequent occurrence in the design of sewerage works is the determination of the economic sizes of force mains. Given two force mains of different diameters for conveying equal quantities of sewage, the one of greater diameter will cost more, but the head due to friction in it and consequently the cost of pumping, will be less than with the pipe of smaller diameter. The most economical diameter of pipe is one that will result in a minimum total cost. It is impossible to determine the exact minimum cost for a term of years, on account of indeterminate losses from friction, variations in costs of coal, pipe and labor, and uncertain changes in the quantity of sewage to be pumped. Making, however, the most reasonable assumptions as to cost of pipe laying and pumping losses from friction, and increase in population, a basis for comparing the relative economy of several pipe lines of different

diameters may be obtained by finding the total annual cost necessary in each case to operate and maintain the structure. This annual cost consists of the following amounts:

1. The annual cost of pumping and repairs, being approximately an average of the annual amounts required during the life of the structure, taking into account the increase in population, friction losses, etc.

2. The annual interest charges on the cost of the property involved.

3. The annual depreciation allowance required. If the pipe line is assumed to have some value at the end of the period under consideration, the depreciation factor is modified so that the depreciation fund amounts at the end of the period to the difference between the first cost and the assumed remaining value of the structure.

While it is possible to deduce a mathematical formula incorporating many of these variable factors, the resulting equation is too complicated to be of much practical value. Furthermore it is necessary to make so many assumptions, in themselves uncertain, that such a formula is of doubtful value when obtained. The most practical way to solve such a problem is by approximation, making two or three assumptions as to the economic velocity and working the results out in detail according to the above cited principles. As an aid to judgment in making these assumptions it is possible to work out a comparatively simple approximate formula as follows:

Let X = diameter of pipe in inches.
Y = cost of pipe and laying per foot in cents.
a = cost of cast iron in cents per pound.
V = velocity in feet per second.
Q = quantity flowing in cubic feet per second.

An examination of the costs of laying cast-iron pipe in different places and under various conditions, indicates that this cost may be represented roughly by the formula

$$Y = 20 + 2.3\,aX^{3/2} \qquad (1)$$

Hazen and Williams' formula for the velocity of flow in pipes, is:

$$V = cR^{0.63}\,S^{0.54}\,0.001^{-0.04} \qquad (2)$$

When $c = 100$ in the Hazen formula, which is the value recommended for use under ordinary conditions for pipe which have been in use for some time, by reduction the following formula may be obtained; D being the diameter in feet to correspond with R, the hydraulic radius, which is in feet in equation 2:

$$V = 55\,D^{0.63}\,S^{0.54} \qquad (3)$$

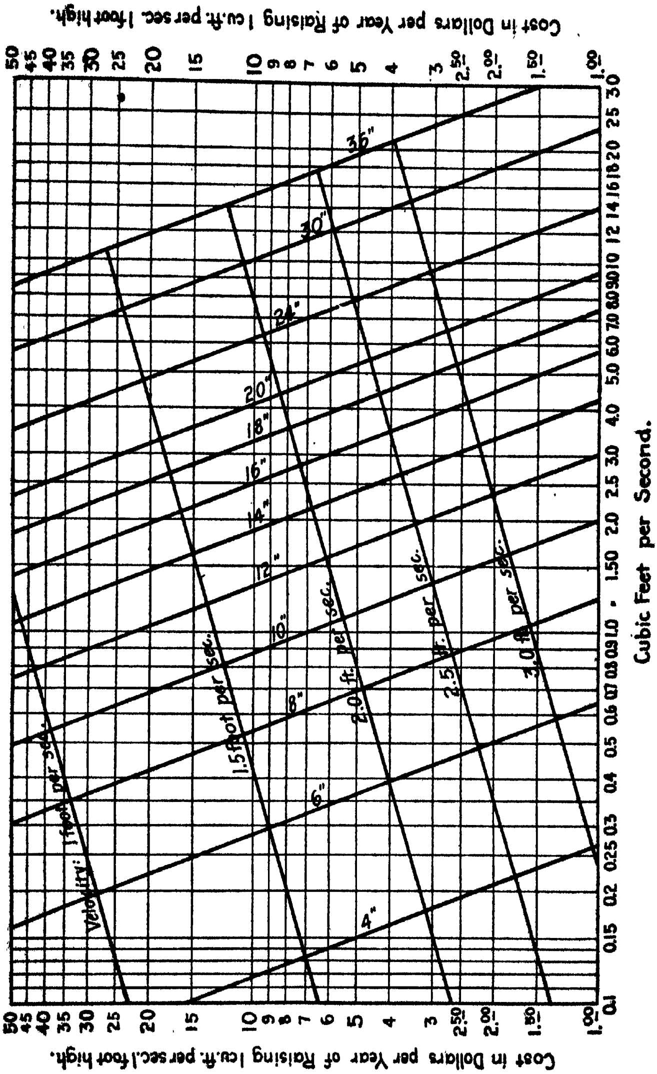


FIG. 327.—Diagram for determining best size of force mains.

If the diameter is given in inches, (3) becomes:

$$V = 11.5\, X^{0.63}\, S^{0.54} \qquad (4)$$

also

$$Q = \pi X^2 V \div (4 \times 144) \qquad (5)$$

Combining and reducing (4) and (5) there results

$$S = 167\, Q^{1.85} \div X^{4.87} \qquad (6)$$

In this final formula S represents the loss of head in feet per foot of pipe.

Let b = cost of pumping 1 cu. ft. per second 1 ft. high per year.
E = fractional part of day during which pumps are operated.
R = rate of interest plus depreciation or sinking fund charges to retire investment cost at end of stated period.

Then the annual cost of pumping per foot of pipe is represented by $E\, b\, Q\, S$, or substituting the value of S from (6)

$$167\, Eb\, Q^{2.85} \div X^{4.87} \qquad (7)$$

and the total annual cost of pipe line and pumping per linear foot of pipe, is represented by the formula,

$$(167\, E\, b\, Q^{2.85} \div X^{4.87}) + 20\, R + 2.3\, aRX^{3/2} \qquad (8)$$

Differentiating this expression with respect to X and placing the result equal to zero, for the purpose of determining the minimum value of X, gives the following result:

$$X = 2.36 \left(\frac{Eb}{aR}\right)^{0.16} Q^{0.45} \qquad (9)$$

If it be assumed that the cost of cast iron is 1.5 cents per pound, that the pumps are operated 24 hours per day, and that interest and sinking fund charges amount to 7 per cent,. the following formula results:

$$Q = 0.067\, X^{2.22} \div b^{0.36} \qquad (10)$$

This formula has been used in the construction of Fig. 327, from which it is possible to determine the approximate economical size of force mains. If assumptions are made, other than those used in preparing the diagram, with regard to the fraction of a day during which pumps are operated and the cost of cast iron, the diagram may still be used, with certain corrections. It will be seen that the diameter varies inversely as $a^{0.16}$, a being the cost of cast iron, so that if the cost is 1.25 cents per pound, the diameter obtained from the diagram should be multiplied by 1.03, and if the price of cast iron is 1 cent per pound, the diameter should be multiplied by 1.07. Similarly the diameter varies

directly as $E^{0.16}$, so that if instead of operating 24 hours a day the pumps are only operated 10 hours, the diameter as determined from the diagram should be multiplied by 0.87. Correction can be similarly made for other values of R. The diagram is based on an assumed coefficient of 100 in the Hazen and Williams formula. If it is desired to base the computation on a factor of 130 or for practically new pipe, the diameter as found in the diagram should be reduced 7 per cent. This formula and diagram do not apply to materials other than cast-iron pipe, but approximations may be made, as for instance where wood stave or steel pipe is used, by determining such a value of a, or cost of cast iron, as would give the correct cost of some one size of the main when built of the material under consideration.

The formula and diagram are based on the assumption that any small change in the diameter of the pipe in order to arrive at the most economical size would not involve any change in the pumping station or pumps, but would involve only differences in costs of pumping based on average unit values. About the only factor of importance involved is the cost of fuel.

In long pipe lines, however, any change in diameter of pipe might involve changes in all the components of the plant, size of station, pumps, cost of operation, etc. Another method of analysis of this problem leading to practically the same results is given on page 603 of Turneaure and Russell's "Public Water Supply," second edition.

STORAGE BASINS ON TIDE WATER

Where the pumping plants are located on tide water, it is occasionally necessary to provide tanks or reservoirs into which the sewage can be delivered while the tidal currents would carry it to places where it would cause a nuisance. The first noteworthy example of such storage in the United States was the reservoir built on Moon Island in 1883, as part of the Boston main drainage works. This had four basins holding 25,000,000 gal., and was designed to store during a period of about 10 hours the sewage pumped to the Island by the Old Harbor Point station, the discharge beginning about an hour after tide commenced to ebb. As usual at that date, the walls of the basins were built of rubble masonry, laid in a 1:2 natural cement mortar, and after some years of service the mortar was found to be very soft. The floors were 9 in., of concrete, the lower 5 in. being made with natural cement and the top 4 in., with Portland cement.

The sewage of the Hampton Institute is discharged into a small tidal stream during the beginning of the ebb tide, and to store it during the remainder of the day tanks were needed. The best site for these was on very low wet ground and the tanks were built above ground, as shown

in Fig. 328 (*Eng. Record*, Nov. 18, 1905). Each is 26-1/2ft. in diameter, inside, and 13 ft. 4 in. high to the springing line of the dome roof, which has a rise of 4-1/2 ft. The tanks are 46-1/2 ft. apart on centers and between them is a ventilating shaft 50 ft. high, with which they are connected, above their flow line, by ducts. Two steam coils in the bottom of the stack, supplied from a neighboring power house, increase the draft.

The entire construction is of 1:2:4, concrete, reinforced where necessary.

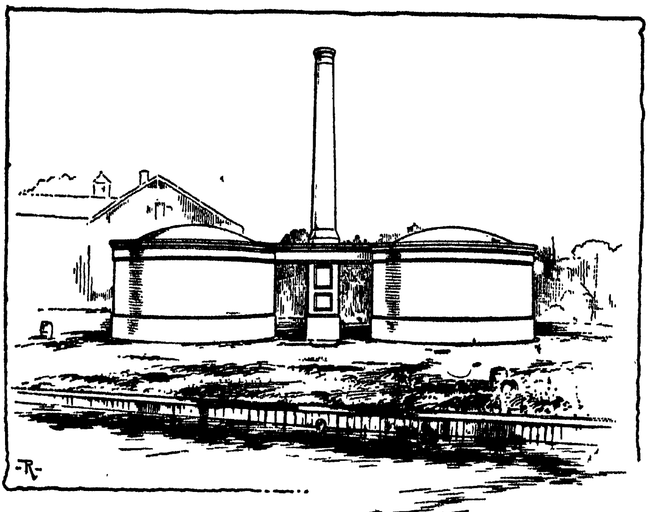


FIG. 328.—Storage tanks at Hampton Institute.

The walls of the tanks are 12 in. thick, and were plastered on the outside and then given three coats of white water paint. The thrust of the dome is taken by a circular iron ring. Sewage is pumped into each tank through two 5-in. pipes, which rise vertically against the wall to a height of nearly 13 ft. above the bottom, thus keeping a uniform head on the pumps. The sewage is drawn off through two 10-in. discharge pipes, emptying into the creek at different places, and the two tanks, which hold about 100,000 gal., can be discharged in 1-1/2 hours.

Index to Tables

Index to Illustrations

The number of the table or illustration is to be found by taking the unit figure in the horizontal line at the top of each table and the tens and hundreds figures in the left-hand column. In the space corresponding to the intersection of the vertical and horizontal lines will be found the page on which the table or illustration appears. For example, Table 47 is on page 168. The letter *i* indicates a folding plate facing the page bearing that number.

INDEX

C

I

L

M

O

P

Y